SPECIAL PUBLICATION 117

GUIDELINES FOR EVALUATING AND MITIGATING SEISMIC HAZARDS IN CALIFORNIA

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DEPARTMENT OF CONSERVATION

Division of Mines and Geology

RCES AGENCY
P. WHEELER
OR RESOURCES

STATE OF CALIFORNIA
PETE WILSON
GOVERNOR

DEPARTMENT OF CONSERVATION
LAWRENCE J. GOLDZBAND
DIRECTOR

DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, *STATE GEOLOGIST*

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Adopted March 13, 1997 by the State Mining and Geology Board in Accordance with the Seismic Hazards Mapping Act of 1990

Copies of these Guidelines, California's Seismic Hazard Mapping Act, and other related information are available on the World Wide Web at http://www.consrv.ca.gov/dmg/shezp/ Copies also are available for purchase from the Public Information Offices of the Division of Mines and Geology.

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Southern California Regional Office
107 South Broadway, Rm. 1065
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(213) 620-3560

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801 K Street, MS 14-33
Sacramento, CA 95814-3532
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Bay Area Regional Office
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(415) 904-7707

CONTENTS

ILLUSTRATIONS

ACKNOWLEDGMENTS

Many people contributed their time and effort to the creation of this document. An asterisk (*) indicates that the listed contributor participated in the past and is no longer a committee or board member.

State Mining and Geology Board:

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Allan Porush, Dames & Moore
Richard Roth, Department of Insurance
William U. Savage, Pacific Gas and Electric
J. Carl Stepp, Electric Power Research Institute*

Amplified Ground Motion Working Group:

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Ken Topping, Consultant*

Division of Mines and Geology Staff:

James F. Davis, State Geologist
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Timothy McCrink, Senior Engineering Geologist
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Robert Sydnor, Senior Engineering Geologist
Earl Hart, Senior Geologist
Mark DeLisle, Associate Engineering Geologist
Mark Petersen, Associate Seismologist
Bea McKamey, Land and Water Use Analyst

CHAPTER 1

INTRODUCTION

Prompted by damaging earthquakes in northern and southern California, in 1990 the State Legislature passed the Seismic Hazards Mapping Act. The Governor signed the Act, codified in the Public Resources Code as Division 2, Chapter 7.8 (see Appendix A), which became operative on April 1, 1991.

The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes. The program and actions mandated by the Seismic Hazards Mapping Act closely resemble those of the Alquist-Priolo Earthquake Fault Zoning Act (which addresses only surface fault-rupture hazards) and are outlined below:

1. **The State Geologist** is required to delineate the various "seismic hazard zones."

2. **Cities and Counties**, or other local permitting authority, must regulate certain development "projects" within the zones. They must withhold the development permits for a site within a zone until the geologic and soil conditions of the project site are investigated and appropriate mitigation measures, if any, are incorporated into development plans.

3. **The State Mining and Geology Board** provides additional regulations, policies, and criteria, to guide cities and counties in their implementation of the law (see Appendix B). The Board also provides guidelines for preparation of the Seismic Hazard Zone Maps (available at http://www.consrv.ca.gov/dmg/shezp/zoneguid.html) and for evaluating and mitigating seismic hazards (this document).

4. **Sellers (and their agents)** of real property within a mapped hazard zone must disclose that the property lies within such a zone at the time of sale.

This document constitutes the guidelines for evaluating seismic hazards other than surface fault-rupture, and for recommending mitigation measures as required by Public Resources Code Section 2695(a). Nothing in these Guidelines is intended to conflict with or supersede any requirement, definition, or other provision of Chapter 7.8 of the Public Resources Code; California Code of Regulations, Title 14, Division 2, Chapter 8, Article 10; the Business and Professions Code; or any other state law or regulation.

Objectives

The objectives of these Guidelines are twofold:

1. To assist in the evaluation and mitigation of earthquake-related hazards for projects within designated zones of required investigations; and

2. To promote uniform and effective statewide implementation of the evaluation and mitigation elements of the Seismic Hazards Mapping Act.

The Guidelines will be helpful to the owner/developer seeking approval of specific development projects within zones of required investigation and to the engineering geologist and/or civil engineer who must investigate the site and recommend mitigation of identified hazards. They will also be helpful to the lead agency engineering geologist and/or civil engineer who must complete the technical review, and other lead agency officials involved in the planning and development approval process. Effective evaluation and mitigation ultimately depends on the combined professional judgment and expertise of the evaluating and reviewing professionals.

The methods, procedures, and references contained herein are those which the State Mining and Geology Board, the Seismic Hazards Mapping Act Advisory Committee, and its Working Groups believe are currently representative of quality practice. Seismic hazard assessment and mitigation is a rapidly evolving field and it is recognized that additional approaches and methods will be developed. If other methods are used, they should be justified with appropriate data and documentation.

For a general description of the Department's Seismic Hazards Mapping Program, its products and their uses, refer to the User's Guide (available in draft form on the World-Wide Web at http://www.consrv.ca.gov/dmg/shezp/userguid.html). A hard-copy edition of the User's Guide will be available later in 1997.

CHAPTER 2

DEFINITIONS, CAVEATS, AND GENERAL CONSIDERATIONS

Definitions

Key terms that will be used throughout the Guidelines are defined in the Act and related regulations. These are:

- **"Seismic Hazards Mapping Act"**— California Public Resources Code Sections 2690 and following, included as Appendix A.

- **"Seismic Hazards Mapping Regulations"**— California Code of Regulations (CCR), Title 14, Division 2, Chapter 8, Article 10, included as Appendix B.

- **"Owner/Developer"** is defined as the party seeking permits to undertake a "project", as defined below.

- **"Mitigation"** means those measures that are consistent with established practice and reduce seismic risk to "acceptable levels" [Public Resources Code (PRC) Section 2693(c)].

- **"Acceptable level"** of risk means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project [CCR Title 14, Section 3721(a)].

- **"Lead agency"** means the state agency, city, or county with the authority to approve projects [CCR Title 14, Section 3721(b)].

- **"Certified Engineering Geologist"** means an engineering geologist who is certified in the State of California [CCR Title 14, Section 3721(c); Business and Professions Code (BPC) Sections 7804 and 7822] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as "engineering geologists." See page 8 *(Engineers or Geologists— Who Does What?)* for a discussion of scope of involvement in site-investigation reports and related reviews.

- **"Registered Civil Engineer"** means a civil engineer who is registered in the State of California [CCR Title 14, Section 3721(c); BPC Sections 6701-6704] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as

"civil engineers." See page 8 *(Engineers or Geologists— Who Does What?)* for a discussion of scope of involvement in site-investigation reports and related reviews.

- **"Site-Investigation Report"** means a report prepared by a certified engineering geologist and/or a civil engineer practicing within the area of his or her competence, which documents the results of an investigation of the site for seismic hazards and recommends mitigation measures to reduce the risk of identified seismic hazards to acceptable levels. In PRC Section 2693(b) and elsewhere, this report is referred to as a "geotechnical report."

- The term **"Project"** is defined by the Seismic Hazards Mapping Act as any structures for human occupancy, or any subdivision of land which contemplates the eventual construction of structures for human occupancy. Unless lead agencies impose more stringent requirements, single-family frame dwellings are exempt unless part of a development of four or more dwellings. (The definition is complex; see Table 1 for specific language.)

- **"Seismic Hazard Zone Maps"** are maps issued by the State Geologist under PRC Section 2696 that show zones of required investigation.

- **"Seismic Hazard Evaluation Reports"** document the data and methods used by the State Geologist to develop the **"Seismic Hazard Zone Maps."**

- **"Zones of Required Investigation,"** referred to as **"Seismic Hazard Zones"** in CCR Section 3722, are areas shown on Seismic Hazard Zone Maps where site investigations are required to determine the need for mitigation of potential liquefaction and/or earthquake-induced landslide ground displacements.

Definitions of technical terms appear in Appendix C.

Caveats

Minimum Statewide Safety Standard

Based on the above definitions of "mitigation" and "acceptable risk," the Seismic Hazards Mapping Act and related regulations establish a statewide minimum public safety standard for mitigation of earthquake hazards. This means that the minimum level of mitigation for a project should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy, but in most cases, ***not*** to a level of no ground failure at all. However, nothing in the Act, the regulations, or these Guidelines precludes lead agencies from enacting more stringent requirements, requiring a higher level of performance, or applying these requirements to developments other than those that meet the Act's definition of "project."

Areal Extent of Hazard

The Seismic Hazard Zone Maps are developed using a combination of historic records, field observations, and computer-mapping technology. The maps may not identify all areas that have potential for liquefaction, earthquake-induced landsliding, strong ground shaking, and other earthquake and geologic hazards. Although past earthquakes have caused ground failures in only

TABLE 1. Definition of "Project"

Public Resources Code Section 2693.

As used in [Chapter 7.8, the Seismic Hazards Mapping Act]:

(d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621), except as follows:

(1) A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.

(2) "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.

Public Resources Code Section 2621.6.

(a) As used in (Chapter 7.5, the Alquist-Priolo Earthquake Fault Zoning Hazard Act), "project" means either of the following:

(1) Any subdivision of land which is subject to the Subdivision Map Act (Division 2 (commencing with Section 66410) of Title 7 of the Government Code), and which contemplates the eventual construction of structures for human occupancy.

(2) Structures for human occupancy, with the exception of either of the following:

(A) Single-family wood-frame or steel-frame dwellings to be built on parcels of land for which geologic reports have been approved pursuant to paragraph (1).

(B) A single-family wood-frame or steel-frame dwelling not exceeding two stories when that dwelling is not part of a development of four or more dwellings.

(b) For the purposes of this chapter, a mobile home whose body width exceeds eight feet shall be considered to be a single-family wood-frame dwelling not exceeding two stories.

California Code of Regulations Section 3601 (Policies and Criteria of the State Mining and Geology Board, With Reference to the Alquist-Priolo Earthquake Fault Zoning Act).

The following definitions as used within the Act and herein shall apply:

(e) A "structure for human occupancy" is any structure used or intended for supporting or sheltering any use of occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year.

(f) "Story" is that portion of a building included between the upper surface of any floor and the upper surface of the floor next above, except that the topmost story shall be that portion of the building included between the upper surface of the topmost floor and the ceiling or roof above. For the purpose of the Act and this subchapter, the number of stories in a building is equal to the number of distinct floor levels, provided that any levels that differ from each other by less than two feet shall be considered as one distinct level."

a small percentage of the total area zoned, a worst-case scenario of a major earthquake during or shortly after a period of heavy rainfall is something that has not occurred in northern California since 1906, and has not been witnessed in historic times in southern California. The damage from such an event in a heavily populated area is likely to be more widespread than that experienced in the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, or the 1994 Northridge earthquake.

Off-Site Origin of Hazard

The fact that a site lies outside a zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, regardless of the information shown on the Seismic Hazard Zone Maps. The zones do not always include landslide or lateral spread run-out areas. Project sites that are outside of any zone may be affected by ground failure runout from adjacent or nearby sites.

Finally, neither the information on the Seismic Hazard Zone Maps, nor in any technical reports that describe how the maps were prepared and what data were used, is sufficient to serve as a substitute for the required site-investigation reports called for in the Act.

Relationship of these Guidelines to Local General Plans and Permitting Ordinances

Public Resources Code Section 2699 directs cities and counties to "take into account the information provided in available seismic hazard maps" when it adopts or revises the safety element of the general plan and any land-use planning or permitting ordinances. Cities and counties should consider the information presented in these guidelines when adopting or revising these plans and ordinances.

Relationship of these Guidelines to the CEQA Process and Other Site Investigation Requirements

Nothing in these guidelines is intended to negate, supersede, or duplicate any requirements of the California Environmental Quality Act (CEQA) or other state laws and regulations. At the discretion of the lead agency, some or all of the investigations required by the Seismic Hazards Mapping Act may occur either before, concurrent with, or after the CEQA process or other processes that require site investigations.

Some of the potential mitigation measures described herein (e.g., strengthening of foundations) will have little or no adverse impact on the environment. However, other mitigation measures (e.g., draining of subsurface water, driving of piles, densification, extensive grading, or removal of liquefiable material) may have significant impacts. If the CEQA process is completed prior to the site-specific investigation, it may be desirable to discuss a broad range of potential mitigation measures (any that might be proposed as part of the project) and related impacts. If, however, part or all of the site-specific investigation is conducted prior to completion of the CEQA process,

it may be possible to narrow the discussion of mitigation alternatives to only those that would provide reasonable protection of the public safety given site-specific conditions.

For hospitals, public schools, and essential services buildings, more stringent requirements are prescribed by the California Building Code (CCR Title 24). For such structures, the requirements of the Seismic Hazards Mapping Act are intended to complement the CCR Title 24 requirements.

Criteria for Project Approval

The State's minimum criteria required for project approval within zones of required investigation are defined in CCR Title 14, Section 3724, from which the following has been excerpted:

> *"The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:*
>
> *(a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.*
>
> *(b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:*
>
> *(1) Project description.*
>
> *(2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.*
>
> *(3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.*
>
> *(4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.*
>
> *(5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.*
>
> *(c) Prior to approving the project, the lead agency shall independently review the geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation."*

Lead agencies can have other, more stringent criteria for project approval. The State Mining and Geology Board recommends that the official professional Registration or Certification Number and license expiration date of each report preparer be included in the signature block of the

report. In addition, Chapter 3 provides a list of topics that should be addressed in site-investigation reports prepared for liquefaction and/or earthquake-induced landslides.

Engineers or Geologists— Who Does What?

The Act and Regulations state that the site-investigation reports must be *prepared* by a certified engineering geologist *or* registered civil engineer, who must have competence in the field of seismic hazard evaluation and mitigation, and be *reviewed* by a certified engineering geologist *or* registered civil engineer, also competent in the field of seismic hazard evaluation and mitigation. *Although the Seismic Hazards Mapping Act does not distinguish between the types of licensed professionals who may prepare and review the report, the current Business and Professions Code (Geologist and Geophysicist Act, Section 7832; and Professional Engineers Act, Section 6704) restricts the practice of these two professions. Because of the differing expertise and abilities of engineering geologists and civil engineers, the scope of the site-investigation report for the project may require that* ***both*** *types of professionals prepare and review the report, each practicing in the area of his or her expertise.* Involvement of both engineering geologists and civil engineers will generally provide greater assurance that the hazards are properly identified, assessed, and mitigated.

The State Mining and Geology Board recommends that engineering geologists and civil engineers conduct the assessment of the surface and subsurface geological/geotechnical conditions at the site, including off-site conditions, to identify potential hazards to the project. It is appropriate for the civil engineer to design and recommend mitigation measures. It also is appropriate for both engineering geologists and civil engineers to be involved in the implementation of the mitigation measures— engineering geologists to confirm the geological conditions and civil engineers to oversee the implementation of the approved mitigation measures.

CHAPTER 3

OVERVIEW OF INVESTIGATIONS FOR ASSESSING SEISMIC HAZARDS

Introduction

Investigation of potential seismic hazards at a site can be performed in two steps or stages: (1) a preliminary **screening investigation**, and (2) a **quantitative evaluation** of the seismic hazard potential and its consequences. As noted below, it is possible to successfully complete the investigation by skipping one or the other stage. For example, a consultant's screening investigation may find that a previous site-specific investigation, on or adjacent to the project site, has shown that no seismic hazards exist, and that a quantitative evaluation is not necessary. Conversely, a consultant may know from experience that a project site is susceptible to a given hazard, and may opt to forego the screening investigation and start with a quantitative evaluation of the hazard.

Some lead agency reviewers recommend that for large projects the developer's consultant(s) meet with the lead agency technical reviewer prior to the start of the site investigation. This allows the consultant and technical reviewer to discuss the scope of the investigation. Topics of this discussion may include the area to be investigated for various hazards, the acceptability of investigative techniques to be used, on-site inspection requirements, or other local requirements.

Items to Consider in the Site Investigation Study

The following concepts are provided to help focus the site-investigation report:

1. Consultants are encouraged to utilize, if possible, the latest California Department of Conservation, Division of Mines and Geology (DMG) seismic ground-motion parameter data. This information is available in DMG's Seismic Hazard Evaluation Reports. The hazard zone mapping procedure for liquefaction and earthquake-induced landsliding utilizes state-of-the-art probabilistic ground-motion parameters developed jointly by the DMG and the U.S. Geological Survey, and published by the DMG (Petersen and others, 1996).

2. The fact that a site lies within a mapped zone of required investigation does not necessarily indicate that a hazard requiring mitigation is present. Instead, it indicates that regional (that is, not site-specific) information suggests that the probability of a hazard requiring mitigation is great enough to warrant a site-specific investigation. However, the

working premise for the planning and execution of a site investigation within Seismic Hazard Zones is that ***the suitability of the site should be demonstrated***. This premise will persist until either: (a) the site investigation satisfactorily demonstrates the absence of liquefaction or landslide hazard, or (b) the site investigation satisfactorily defines the liquefaction or landslide hazard and provides a suitable recommendation for its mitigation.

3. The fact that a site lies outside a mapped zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, nor does it preclude lead agencies from adopting regulations or procedures that require site-specific soil and/or geologic investigations and mitigation of seismic or other geologic hazards. It is possible that development proposals may involve alterations (for example, cuts, fills, and/or modifications that would significantly raise the water table) that could cause a site outside the zone to become susceptible to earthquake-induced ground failure.

4. Lead agencies have the right to approve (and the obligation to reject) a proposed project based on the findings contained in the site-investigation report and the lead agency's technical review. The task of the developer's consulting engineering geologist and/or civil engineer is to demonstrate, to the satisfaction of the lead agency's technical reviewer, that:
   - The site-specific investigation is sufficiently thorough;
   - The findings regarding identified hazards are valid; and
   - The proposed mitigation measures achieve an acceptable level of risk, as defined by the lead agency and CCR Title 14, Section 3721(a).

Screening Investigation

The purpose of screening investigations for sites within zones of required investigation is to evaluate the severity of potential seismic hazards, or to screen out sites included in these zones that have a low potential for seismic hazards. If a screening investigation can ***clearly*** demonstrate the absence of seismic hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of seismic hazards, then the more-comprehensive quantitative evaluation needs to be conducted.

The documents reviewed should be both regional and, if information is available, site-specific in scope. The types of information reviewed during a screening investigation often includes topographic maps, geologic and soil engineering maps and reports, aerial photographs, water well logs, agricultural soil survey reports, and other published and unpublished references. The references used should focus on current journals, maps, reports, and methods. Seismic Hazard Evaluation Reports, which summarize the findings and data on which DMG's Seismic Hazard Zone Maps are based, can provide much of the regional geologic and seismic information needed for a screening investigation. Aerial photographs can be useful to identify existing and potential landslide and/or liquefaction features (headwall scarps, debris chutes, fissures, grabens, sand boils, etc.) that suggest or preclude the existence of ground failure hazards that might affect the site. Several sets of stereoscopic aerial photographs that pre-date project site area development, and

taken during different seasons of the year are particularly useful for identifying subtle geomorphic features. A field reconnaissance of the area is highly recommended to verify the information developed in the earlier steps to fill in information in questionable areas, and to observe the surface features and details that could not be determined from other data sources.

Quantitative Evaluation of Hazard Potential

Detailed Field Investigations— General Information Needs

Within the zone of required investigations, the objective of the detailed field investigation is to obtain sufficient information on which the engineering geologist and/or civil engineer can evaluate the nature and severity of the risk and develop a set of recommendations for mitigation. In the case of projects where the property is to be subdivided and sold to others undeveloped, the aim of the investigation is to determine which parcels contain buildable sites that meet the previously defined acceptable level of risk. The work should be based upon a detailed, accurate topographic base map prepared by a registered civil engineer or land surveyor. The map should be of suitable scale, and should cover the area to be developed as part of the project, as well as adjacent areas which affect or may be affected by the project.

The detailed field investigation commonly involves the collection of subsurface information from trenches or borings, on or adjacent to the site. The subsurface exploration should extend to depths sufficient to expose geologic and subsurface water conditions that could affect slope stability or liquefaction potential. A sufficient quantity of subsurface information is needed to permit the engineering geologist and/or civil engineer to extrapolate with confidence the subsurface conditions that might affect the project, so that the seismic hazard can be properly evaluated, and an appropriate mitigation measure can be designed by the civil engineer.

The preparation of engineering geologic maps and geologic cross sections is often an important step to developing an understanding of the significance and extent of potential seismic hazards. These maps and/or cross sections should extend far enough beyond the site to identify off-site hazards and features that might affect the site.

Content of Reports

The site investigation report should contain sufficient information to allow the lead agency's technical reviewer to satisfactorily evaluate the potential for seismic hazards and the proposed mitigation. No attempt is made here to define the limits of what constitutes a complete screening investigation or quantitative evaluation report. Site-specific conditions and circumstances, as well as lead agency requirements, will dictate which issues and what level of detail are required to adequately define and mitigate the hazard(s). The following list (Table 2) is provided to assist investigators and reviewers in identifying seismic hazard-related factors significant to the project. Not all of the information in the list will be relevant nor required, and some investigations may require additional types of data or analyses.

Table 2. Recommended content for site-investigation reports within zones of required investigations.

Reports that address liquefaction and/or earthquake-induced landslides should include, but not necessarily be limited to, the following data:	
1.	Description of the proposed project's location, topographic relief, drainage, geologic and soil materials, and any proposed grading.
2.	Site plan map of project site showing the locations of all explorations, including test pits, borings, penetration test locations, and soil or rock samples.
3.	Description of seismic setting, historic seismicity, nearest pertinent strong-motion records, and methods used to estimate (or source of) earthquake ground-motion parameters used in liquefaction and landslide analyses.
4.	1:24,000 or larger-scale geologic map showing bedrock, alluvium, colluvium, soil material, faults, shears, joint systems, lithologic contacts, seeps or springs, soil or bedrock slumps, and other pertinent geologic and soil features existing on and adjacent to the project site.
5.	Logs of borings, test pits, or other subsurface data obtained.
6.	Geologic cross sections depicting the most critical (least stable) slopes, geologic structure, stratigraphy, and subsurface water conditions, supported by boring and/or trench logs at appropriate locations.
7.	Laboratory test results; soil classification, shear strength, and other pertinent geotechnical data.
8.	Specific recommendations for mitigation alternatives necessary to reduce known and/or anticipated geologic/seismic hazards to an acceptable level of risk.
Reports that address earthquake-induced landslides may also need to include:	
1.	Description of shear test procedures (ASTM or other) and test specimens.
2.	Shear strength plots, including identification of samples tested, whether data points reflect peak or residual values, and moisture conditions at time of testing.
3.	Summary table or text describing methods of analysis, shear strength values, assumed groundwater conditions, and other pertinent assumptions used in the stability calculations.
4.	Explanation of choice of seismic coefficient and/or design strong-motion record used in slope stability analysis, including site and/or topographic amplification estimates.
5.	Slope stability analyses of critical (least-stable) cross sections which substantiate conclusions and recommendations concerning stability of natural and as-graded slopes.
6.	Factors of safety against slope failure and/or calculated displacements for the various anticipated slope configurations (cut, fill, and/or natural slopes).
7.	Conclusions regarding the stability of slopes with respect to earthquake-induced landslides and their likely impact on the proposed project.
8.	Discussion of proposed mitigation measures, if any, necessary to reduce damage from potential earthquake-initiated landsliding to an acceptable level of risk.
9.	Acceptance testing criteria (e.g., pseudo-static factor of safety), if any, that will be used to demonstrate satisfactory remediation.
Reports that address liquefaction hazards may also need to include the following:	
1.	If methods other than Standard Penetration Test (SPT; ASTM D1586-92) and Cone Penetration Test (CPT; ASTM 3441-94) are used, description of pertinent equipment and procedural details of field measurements of penetration resistance (borehole type, hammer type and drop mechanism, sampler type and dimensions, etc.),
2.	Boring logs showing raw (unmodified) N-values if SPT's are performed; CPT probe logs showing raw q_c-values and plots of raw sleeve friction if CPT's are performed.
3.	Explanation of the basis and methods used to convert raw SPT, CPT, and/or other non-standard data to "corrected" and "standardized" values.
4.	Tabulation and/or plots of corrected values used for analyses.
5.	Explanation of methods used to develop estimates of field loading equivalent uniform cyclic stress ratios (CSR_{eq}) used to represent the anticipated field earthquake excitation (cyclic loading).

Table 2. Recommended content for site-investigation reports within zones of required investigations.	
6.	Explanation of the basis for evaluation of the equivalent uniform cyclic stress ratio necessary to cause liquefaction (CSR_{liq}) within the number of equivalent uniform loading cycles considered representative of the design earthquake.
7.	Factors of safety against liquefaction at various depths and/or within various potentially liquefiable soil units.
8.	Conclusions regarding the potential for liquefaction and its likely impact on the proposed project.
9.	Discussion of proposed mitigation measures, if any, necessary to reduce potential damage caused by liquefaction to an acceptable level of risk.
10.	Criteria for SPT-based, CPT-based, or other types of acceptance testing , if any, that will be used to demonstrate satisfactory remediation.

CHAPTER 4

ESTIMATION OF EARTHQUAKE GROUND-MOTION PARAMETERS

Introduction

Quantitative analyses of in-situ liquefaction resistance and earthquake-induced landslide potential requires site-specific assessment of ground shaking levels suitable for those purposes. A simplified Seed-Idriss (1982) liquefaction analysis requires an estimation of peak ground acceleration (PGA) and earthquake magnitude. A pseudo-static slope stability analysis may require estimates of PGA and magnitude for the selection of an appropriate seismic coefficient. If a seismic site response analysis is needed, or if a finite element analysis, a Newmark analysis or a dynamic analysis is to be performed, a representative strong-motion record will need to be selected on the basis of site-specific ground-motion parameter estimates. The following sections of this Chapter provide guidance on the selection of site-specific ground-motion parameters and representative strong-motion records.

Simple Prescribed Parameter Values (SPPV)

Probabilistic ground-motion parameter values on firm rock for PGA, predominant magnitude, and distance in the form of statewide maps have been jointly prepared by DMG and the U.S. Geological Survey for a 10 percent probability of exceedance over a 50-year period (Petersen and others, 1996). Versions of these maps covering a 7.5 minute quadrangle area at a scale of 1:100,000 are included in the Seismic Hazard Evaluation Reports that accompany Seismic Hazard Zone Maps. The predominant magnitude and distance maps are not dependent on site conditions, and can be used for site-specific purposes. PGA can be dependent on site conditions and several maps have been prepared to accommodate these differences, each based on site-dependent attenuation relations consistent with the soil profile types identified in the Uniform Building Code (ICBO, 1997). These maps are included in the Seismic Hazard Evaluation Reports issued by DMG, and can be used to obtain PGA as follows:

1. Classify the site according to the procedures and soil profile types defined in Chapter 16 of the Uniform Building Code (ICBO, 1997), and interpolate PGA from the corresponding PGA map; or

2. Interpolate PGA from the representative bedrock PGA map, and apply an appropriate scaling factor based on the soil profile type; or

3. Perform a site response analysis (e.g., using a finite-element or SHAKE program to simulate the effects of ground-motion propagating through a soil column). Bedrock PGA and predominant magnitude and distance obtained from the above maps can be used to select an appropriate strong-motion record for input into the response analysis.

PGA estimated by the above procedures may still require additional adjustment to account for topographic and basin effects. Use of the SPPV method is not recommended for sites located very near to seismic sources, where reliable ground-motion estimates may require consideration of near-field source effects.

Probabilistic Seismic Hazard Analysis (PSHA)

Site-specific probabilistic seismic hazard analyses can be performed, and can supersede the SPPV-values of PGA for seismic hazard studies, even if PSHA studies result in adoption of a lower level of seismic ground motion. PSHA studies typically include the following:

1. A database consisting of potentially damaging earthquake sources, including known active faults and historic seismic source zones, their activity rates, and distances from the project site. This should include a comparison with DMG-developed slip rates for faults considered in the DMG statewide probabilistic seismic hazard map. Differences in slip rates should be documented and the reasons for them explained (for example, revised slip rates or new paleoseismic information from recent studies). DMG recommends that their earthquake source database be used directly, because it is updated regularly and is readily available (Petersen and others, 1996; see the World Wide Web at http://www.consrv.ca.gov/dmg/shezp/);

2. Use of published maximum moment magnitudes for earthquake sources, or estimates that are justified, well-documented, and based on published procedures ;

3. Use of published curves (or those used by DMG) for attenuation of PGA with distance from earthquake source, as a function of earthquake magnitude and travel path (e.g., see special issue of Seismological Research Letters, v. 68, n. 1, 1997);

4. An evaluation of the likely effects of site-specific response characteristics (e.g., amplification due to soft soils, deep sedimentary basins, topography, near-source effects, etc.);

5. Characterization of the ground motion at the site in terms of PGA with a 10 percent probability of exceedance in 50 years, taking into account historical seismicity, available paleoseismic data, the geological slip rate of regional active faults, and site-specific response characteristics.

Useful references include Reiter, 1990; National Research Council, 1988; Hayes, 1985; Algermissen and others, 1982; Cornell, 1968; Youngs and Coppersmith, 1985; Working Group on California Earthquake Probabilities, 1990 and 1995; Okumura and Shinozuka, 1990; and Kramer, 1996.

Deterministic Seismic Hazard Analysis (DSHA)

Deterministic evaluation of seismic hazard can also be performed, and the results of correctly performed and suitably comprehensive DSHA studies can also supersede SPPV values of PGA. DSHA studies typically include the following:

1. Evaluation of potentially damaging earthquake sources, and deterministic selection of one or more suitable "controlling" sources and seismic events. The deterministic earthquake event magnitude for any fault should be a ***maximum*** value that is specific to that seismic source. Maximum earthquakes may be assessed by estimating rupture dimensions of the fault (e.g., Wells and Coppersmith, 1994; dePolo and Slemmons, 1990). The DMG database of earthquake sources is readily available (see section on PSHA);

2. Use of published curves for the effects of seismic travel path using the shortest distance from the source(s) to the site (e.g., see special issue of Seismological Research Letters, v. 68, n.1, 1997);

3. Evaluation of the effects of site-specific response characteristics on either (a) site accelerations, or (b) cyclic shear stresses within the site soils of interest.

Selection of a Site-Specific Design Strong-Motion Record

In the course of performing a seismic slope stability or liquefaction analysis, it is often necessary to select a design strong-motion record that represents the anticipated earthquake shaking at a project site. For a seismic slope-stability analysis the design strong-motion record will be used to evaluate the site seismic response (site amplification) and/or for the calculation of Newmark displacements. In some cases, the strong-motion record will be the input ground motion for a detailed dynamic analysis. For liquefaction evaluations the design strong-motion record will be used for the site seismic response to determine the appropriate peak ground acceleration to use in a simplified Seed-Idriss liquefaction analysis. It could also be used for a detailed finite-element analysis where the magnitude of potential lateral spread displacements are critical to the proposed project.

The selection process typically involves two steps: (1) estimating magnitude, epicentral distance and peak ground acceleration parameters for the project site, and (2) searching for existing strong-motion records that have parameters that closely match the estimated values. The methods described in the preceding sections of this chapter describe the recommended approaches to the parameter estimates. The selection of a representative strong-motion record should consider the following:

1. The selection should be based primarily on matching magnitude, epicentral distance, site conditions and PGA between the site and the record, generally in that order;

2. It may not always be possible to find a good match between the site parameters and the existing strong-motion records, and it may be necessary to use a record that does not match the site parameter criteria and scale it to fit those parameters, making sure that the duration of the scaled record is appropriate for the anticipated magnitude;

3. If the site to be analyzed is underlain by soils or weakly cemented rock, and a strong-motion recording site with similar characteristics cannot be found, a seismic site response analysis should be performed (e.g., SHAKE91, Idriss and Sun, 1992; SHAKE, Schnabel and others, 1972);

4. For project sites that could experience earthquakes from both high-frequency, near-source events and low-frequency, long-duration events, multiple records representative of these events should be included in the analysis.

A database of strong-motion records is available at the DMG World Wide Web site { http://www.consrv.ca.gov/dmg/ }. This and other sources for acquiring strong-motion records are provided in Appendix D.

CHAPTER 5

ANALYSIS AND MITIGATION OF EARTHQUAKE-INDUCED LANDSLIDE HAZARDS

Screening Investigations for Earthquake-Induced Landslide Potential

The purpose of screening investigations for sites within zones of required investigation for earthquake-induced landslides is to evaluate the severity of the hazard, or to screen out sites included in these zones that have a low potential for landslide hazards. If a screening investigation can ***clearly*** demonstrate the absence of earthquake-induced landslide hazard at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of the hazard, then the more-comprehensive quantitative evaluation needs to be conducted.

An important aspect of evaluating the potential for earthquake-induced landslides is the recognition of the types of slope failures commonly caused by earthquakes. Keefer (1984) studied 40 historical earthquakes and found that different types of landslides occur with different frequencies. Table 3 summarizes Keefer's findings. In addition, Keefer (1984) summarized the geologic environments that are likely to produce earthquake-induced landslides. A table of these environments is provided in Appendix E to assist in the evaluation of project sites for the screening investigations.

The screening investigation should evaluate, and the report should address, the following basic questions:

- **Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?**

  An assessment of the presence of existing landslides on the project site for a screening investigation will typically include a review of published and unpublished geologic and landslide inventory maps of the area and an interpretation of aerial photographs. The distinctive landforms associated with landslides (scarps, troughs, disrupted drainages, etc.)

Table 3. Relative abundance of earthquake-induced landslides from 40 historical earthquakes (Keefer, 1984; Table 4, p. 409).

Relative Abundance of Earthquake-Induced Landslides	Description
Very Abundant (more than 100,000 in the 40 earthquakes)	Rock falls, disrupted soil slides, rock slides
Abundant (10,000 to 100,000 in the 40 earthquakes)	Soil lateral spreads, soil slumps, soil block slides, soil avalanches
Moderately common (1000 to 10,000 in the 40 earthquakes)	Soil falls, rapid soil flows, rock slumps
Uncommon (100 to 1000 in the 40 earthquakes)	Subaqueous landslides, slow earth flows, rock block slides, rock avalanches

should be noted, if present, and the possibility that they are related to landslides should be assessed.

- **Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?**

  Many geologic formations in California, notably late Tertiary and Quaternary siltstones and shales (for example, the Orinda and Modelo formations), are highly susceptible to landsliding. These rock units are generally well-known among local engineering geologists. For some areas, susceptible formations have also been noted on the Landslide Hazard Identification Maps published by DMG.

- **Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?**

  Subsurface water in slopes can be an important indicator of landslide potential. Water may be forced to the surface along impermeable layers such as landslide rupture surfaces. Springs, seeps, or vegetation (phreatophytes) may result from impermeable layers and near-surface water. Topographic depressions, heavy irrigation, or disrupted surface water channels can cause ponding and increased infiltration of surface water. These features may be visible on pre- and/or post-development aerial photographs taken during certain seasons, or during a field reconnaissance. Presence of shallow subsurface water is significant because pore-water pressure reduces the forces resisting landslide movement.

- **Are susceptible landforms and vulnerable locations present? These include steep slopes, colluvium-filled swales, cliffs or banks being undercut by stream or wave action, areas that have recently slid.**

  In addition to existing landslide deposits, certain other slopes are especially susceptible to landsliding. These include very steep slopes, and ones where the support at the base of the slope has been removed or reduced. Removal of support at the base of a slope occurs naturally by stream or wave erosion and the same effect can be produced by grading of cut slopes. Colluvium-filled swales usually develop naturally over thousands of years, and the resulting thick, deeply weathered soil may be especially susceptible to debris flows. Hazardous slope features can generally be noted on aerial photographs, sufficiently detailed topographic maps, or from a geologic field reconnaissance.

- **Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?**

  Misdirected runoff from streets during rainstorms can cause saturation of surficial materials and, in turn, development of catastrophic debris flows. Improperly designed highway culverts and watering of lawns on marine terraces can create unstable gullies, undermined coastal bluffs, or both. It is likely that the proposed development will alter the local groundwater regime in some way. The investigation should describe the likely effects that altered runoff patterns, lawn watering or septic systems will have on slope stability; identify sensitive areas; and, when warranted, recommend mitigation.

Additional Considerations

It is recommended by the Earthquake-Induced Landslides Working Group that the screening investigation should include a site reconnaissance by the project engineering geologist and/or civil engineer. This will allow for the recognition of potential earthquake hazards that cannot normally be recognized in a purely office-based screening investigation.

Guidance on the preparation of a report for the screening investigation is provided in Chapter 3 of these Guidelines. If the results of the screening investigation show that the potential for earthquake-induced landsliding is low, the report should state the reasons why a quantitative evaluation is not needed for the project site.

Quantitative Evaluation of Earthquake-Induced Landslide Potential

If the screening investigation indicates the presence of potentially unstable slopes affecting the proposed project site, a quantitative evaluation of earthquake-induced landslide potential should be conducted. The major phases of such a study typically includes a detailed field investigation, drilling and sampling, geotechnical laboratory testing, and slope stability analyses. Reference should be made to Chapter 3 for guidance on what types of information from the following sections should be included in the site-investigation report.

Detailed Field Investigation

Engineering Geologic Investigations

The engineering geologic investigation phase of the project site investigation consists of surface observations and geologic mapping. The overall scope of the engineering geologic investigation for earthquake-induced landslide hazards is fundamentally the same as the work that would be conducted for any project that has potential landslide hazards, regardless of the triggering mechanism. However, the investigator should keep in mind the environments and the relative abundance of landslide types triggered by earthquakes as described by Keefer (1984) and shown in Appendix E and Table 3, respectively. The engineering geologic investigation is significant because it provides the basis for the subsurface investigations, field instrumentation, and geotechnical analyses that follow.

Prior to the site reconnaissance, the area of the project should be identified, and available geologic and geotechnical information, stereoscopic aerial photographs, and topographic maps should be collected and reviewed (Keaton and DeGraff, 1996). If a screening investigation has been conducted for the site, much of this information may already have been reviewed. Once the results of the office-based investigation have been completed and understood, on-site engineering geologic mapping can be conducted.

The purpose of the on-site engineering geologic mapping is to document surface conditions which, in turn, provides a basis for projecting subsurface conditions that may be relevant to the stability of the site. The on-site engineering geologic mapping should identify, classify, and locate on a map the features and characteristics of existing landslides, and surficial and bedrock geologic materials. Other important aspects of the site to document include: landslide features and estimates of depth to the rupture surface; distribution and thickness of colluvium; rock discontinuities such as bedding, jointing, fracturing and faulting; depth of bedrock weathering; surface water features such as streams, lakes, springs, seeps, marshes, and closed or nearly closed topographic depressions.

Engineering geologic cross sections should be located so as to provide information that will be needed for planning subsurface investigations and stability analyses. The most useful orientation is typically perpendicular to topographic contours and longitudinally down existing landslide deposits. The projected shape of the rupture surface, geologic contacts and orientations, and groundwater surfaces should be shown along with the topographic profile. Estimates of the depth to the landslide rupture surface is an important parameter for planning a subsurface investigation and longitudinal cross sections can be helpful in making these estimates (McGuffey and others, 1996).

The results of the engineering geologic mapping can be presented in many forms, but generally should include a map, cross sections, and proposed subsurface investigation locations and/or field instrumentation sites. Whatever method of presentation is chosen, it should be remembered that the presentation of the surface mapping information needs to be characterized in terms that are meaningful for, and usable by the design engineer. Doing so will help ensure that key factors that must be accommodated in the construction are understood (Keaton and DeGraff, 1996).

Subsurface Investigation

Planning

Exploratory work by the engineering geologist and civil engineer should be conducted at locations considered most likely to reveal any subsurface conditions which may indicate the potential for earthquake-induced landslide failures. In particular, an investigation should locate and define the geometry of bedding and fracture surfaces, contacts, faults, and other discontinuities as well as actual landslide rupture surfaces.

Subsurface exploration methods can be classed as direct methods and indirect methods (Hunt, 1984a). Direct methods, such as test borings and the excavation of test pits or trenches, allow the examination of the earth materials, usually with the removal of samples. Indirect methods, such as geophysical surveys and the use of the cone penetrometer, provide a measure of material properties that allows the estimation of the material type (McGuffey and others, 1996).

Subsurface investigations should be supervised by an experienced engineering geologist and/or civil engineer to ensure that the field activities are properly executed and the desired results are achieved. According to McGuffey and others (1996), the subsurface investigation field supervision should:

1. Ensure that technical and legal contract specifications are followed;
2. Maintain liaison with the designer of the exploration program;
3. Select and approve modifications to the program as new or unanticipated conditions are revealed;
4. Ensure that complete and reliable field reports are developed; and
5. Identify geologic conditions accurately.

The depth to which explorations should extend can be difficult to define in advance of the subsurface investigation. Cross sections from a surface engineering geological investigation can be helpful in planning the depths of excavations required in a subsurface investigation. In general, borings or other direct investigative techniques should extend deep enough (a) to identify materials that have not been subjected to movements in the past but might be involved in future movements, and (b) to clearly identify underlying stable materials. The exploration program plan should be flexible enough to allow for expanding the depth of investigation when the data obtained suggest deeper movements are possible (McGuffey and others, 1996).

Samples and Sampling

Soil and rock samples that may be obtained from subsurface borings and excavations belong to one of two basic categories: disturbed and undisturbed samples. Disturbed samples are collected primarily for soil classification tests where the preservation of the soil structure is not essential, or for remolding in the laboratory and subsequent strength and compressibility tests. Undisturbed samples do not entirely represent truly undisturbed soil or rock conditions because the process of sampling and transporting inevitably introduces some disturbance into the soil or rock structure.

These samples are taken primarily for laboratory strength and compressibility tests and for the measurement of in-situ material properties.

Samples of the soil, the existing landslide rupture materials, and the weakest components of rock units should be taken for laboratory measurement of engineering properties. Special care should be taken to obtain oriented samples of existing zones of weakness or rupture surfaces. For shallow landslides it may be possible to expose and sample critical zones of weakness in the walls of trenches or test pits. For deep-seated landslides it often is extremely difficult to sample the zones of weakness with typical geotechnical drilling equipment, and it may be appropriate to consider using bucket auger drilling and down-hole geologic logging and sampling techniques (Scullin, 1994).

It is the responsibility of the field supervising geologist or engineer to accurately label and locate the collected samples. They are also responsible for the proper transportation of collected samples, particularly undisturbed samples, to prevent sample disturbance by excessive shocks, allowing samples to dry or slake, or by exposing samples to heat or freezing conditions. A large variety of soil boring techniques and sampler types is available. A detailed explanation of the many types is beyond the scope of these Guidelines, but are readily available in the literature (Hvorslev, 1948; ASTM, 1971 and 1997; U.S. Bureau of Reclamation, 1974 and 1989; U.S. Navy, 1986; Hunt, 1984a; Krynine and Judd, 1957; Acker, 1974; Scullin, 1994; Johnson and DeGraff, 1988; McGuffey and others, 1996).

Subsurface Water

The presence of subsurface water can be a major contributing factor to the dynamic instability of slopes and existing landslides. Therefore, the identification and measurement of subsurface water in areas of suspected or known slope instability should be an integral part of the subsurface investigation. The location and extent of groundwater, perched groundwater and potential water barriers should be defined. Subsurface water conditions within many landslides are best considered as complex, multiple, partially connected flow systems. McGuffey and others (1996) have listed the following recommendations:

1. Surface observations are essential in determining the effect of subsurface water on landslide instability.

2. Periodic or seasonal influx of surface water to subsurface water will not be detected unless subsurface water observations are conducted over extended time periods.

3. Landslide movements may open cracks and develop depressions at the head of a landslide that increase the rate of infiltration of surface water into the slide mass.

4. Ponding of surface water anywhere on the landslide may cause increased infiltration of water into the landslide and should be investigated.

5. Disruption of surface water channels and culverts may also result in increased infiltration of surface water into the landslide.

6. Landslide movements may result in blockage of permeable zones that were previously freely draining. Such blockage may cause a local rise in the groundwater table and increased saturation and instability of the landslide materials. Subsurface observations should therefore be directed to establishing subsurface water conditions in the undisturbed areas surrounding the landslide.

7. Low permeability soils, which are commonly involved in landslides, have slow response times to changes in subsurface water conditions and pressures. Long-term subsurface water monitoring is required in these soils.

8. Accurate detection of subsurface water in rock formations is often difficult because shale or claystone layers, intermittent fractures, and fracture infilling may occlude subsurface water detection by boring or excavation.

9. Borings should never be the only method of subsurface water investigation; nevertheless they are a critical component of the overall investigation.

Geotechnical Laboratory Testing

The geotechnical testing of soil and rock materials typically follows accepted published standards (ASTM, 1997; Head, 1989). Good professional judgment is expected in the selection of appropriate samples, shear tests and interpretation of the results in arriving at strength characteristics appropriate for the present and anticipated future slope conditions. The following guidelines are provided for evaluating soil properties.

1. Soil properties, including unit weight and shear strength parameters (cohesion and friction angle), may be based on appropriate conventional laboratory and field tests.

2. Testing of earth materials should be in accordance with the appropriate ASTM Standards that are updated annually (ASTM, 1997).

3. Prior to shear tests, samples should be soaked a sufficient length of time to approximate a saturated moisture condition.

4. Stability analyses generally should use the lowest values derived from the suite of samples tested.

5. Residual test values should be used for static analysis of existing landslides, along shale bedding planes, highly distorted bedrock, over-consolidated fissured clays, and for paleosols and topsoil zones under fill. Peak values may be used for pseudo-static or dynamic calculations if the buildup of pore pressures is not anticipated and if permitted by the lead agency. Consideration of reducing the strength values for dynamic analyses should be made in light of the measured material properties and anticipated subsurface water conditions (see section on Effective-Stress vs. Total-Stress Conditions below).

6. Appropriate analyses of existing failures (back-calculated strengths) in slopes similar to that under consideration in terms of height, geology, and soil or rock materials may be helpful in determining or verifying proposed shear strength parameters.

7. Laboratory shear strength values used for design of fill slopes steeper than two horizontal to one vertical (2:1) and for buttress fills should be verified by testing during slope grading. In the event that the shear strength values from field samples are less than those used in design, the slope should be reanalyzed and modified as necessary to provide the required factor of safety for stability.

Slope Stability Analysis

General Considerations

Slope stability analysis will generally be required by the lead agency for cut, fill, and natural slopes whose slope gradient is steeper than two horizontal to one vertical (2:1), and on other slopes that possess unusual geologic conditions such as unsupported discontinuities or evidence of prior landslide activity. Analysis generally includes deep-seated and surficial stability evaluation under both static and dynamic (earthquake) loading conditions.

Evaluation of deep-seated slope stability should be guided by the following:

1. The potential failure surface used in the analysis may be composed of circles, planes, wedges or other shapes considered to yield the minimum factor of safety against sliding for the appropriate soil or rock conditions. The potential failure surface having the lowest factor of safety should be sought.

2. Forces to be considered include the gravity loads of the potential failure mass, structural surcharge loads and supported slopes, and loads due to anticipated earthquake forces. The potential for hydraulic head (or significant pore-water pressure) should be evaluated and its effects included when appropriate. Total unit weights for the appropriate soil moisture conditions are to be used.

Evaluation of surficial slope stability should be guided by the following:

1. Calculations may be based either on analysis procedures for stability of an infinite slope with seepage parallel to the slope surface or on another method acceptable to the lead agency. For the infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one (1) meter or depth to firm bedrock. Soil strength characteristics used in analysis should be obtained from representative samples of surficial soils that are tested under conditions approximating saturation and at normal loads approximating conditions at very shallow depth.

2. Appropriate mitigation procedures and surface stabilization should be recommended, in order to provide the required level of surficial slope stability.

3. Recommendations for mitigation of damage to the proposed development caused by failure of off-site slopes should be made unless slope-specific investigations and analyses demonstrate that the slopes are stable. Ravines, swales, and hollows on natural slopes warrant special attention as potential sources of fast-moving debris flows and other types of landslides. If possible, structures should be located away from the base or axis of these types of features.

Analysis Methods Available

There are four generally accepted methods of slope stability analysis for seismic loading conditions. Two of these methods, the pseudo-static analysis and the Newmark analysis, have practical applications for most residential and commercial development projects affected by Seismic Hazard Zone Maps, and will be discussed in some detail in the following sections. The other two methods, the Makdisi-Seed (1978) analysis and the dynamic analysis, are not generally applicable to these types of developments. These latter two methods will only be briefly summarized in this section.

The simplest approach to a dynamic slope stability calculation is the **pseudo-static analysis** in which the earthquake load is simulated by an "equivalent" static horizontal acceleration acting on the mass of the landslide in a limit-equilibrium analysis (Nash, 1987; Janbu, 1973; Bromhead, 1986; Chowdhury, 1978; Morgenstern and Sangrey, 1978; Hunt, 1984b; Duncan, 1996). The pseudo-static approach has certain limitations (Cotecchia, 1987; Kramer, 1996), but this methodology is considered to be generally conservative, and is the one most often used in current practice.

The second procedure is known as the **Newmark or cumulative displacement analysis** (Newmark, 1965; Makdisi and Seed, 1978; Hynes and Franklin, 1984; Houston and others, 1987; Wilson and Keefer, 1983; Jibson, 1993). The procedure involves the calculation of the yield acceleration, defined as the inertial force required to cause the static factor of safety to reach 1.0, from the traditional limit-equilibrium slope stability analysis. The procedure then uses a design earthquake strong-motion record which is numerically integrated twice for the amplitude of the acceleration above the yield acceleration to calculate the cumulative displacements. These analytical displacements are then evaluated in light of the slope material properties and the requirements of the proposed development. The pseudo-static and Newmark analyses will be described in more detail in the following sections.

The third method is referred to as a **Makdisi-Seed analysis** (Makdisi and Seed, 1978; Kramer, 1996). Makdisi and Seed's work (1978) sought to define seismic embankment stability in terms of acceptable deformations in lieu of conventional factors of safety, using a modified Newmark analysis. Their method presents a rational means by which to determine yield acceleration, or the average acceleration required to produce a factor of safety of unity. This value, in turn is affected by the cyclic-yield strengths of embankment materials, which turned out to be about 80 percent of static strength. Design curves were developed to estimate the permanent earthquake-induced deformations of embankments 100 to 200 feet high using finite element analyses. These same methods have since been applied to sanitary landfill and highway embankments. Very little application of this method has been made to pre-existing landslides, and the method will not be reviewed in detail in these guidelines.

The most sophisticated method for seismic slope stability calculations is known as a **dynamic analysis** (Cotecchia, 1987) or a **stress-deformation analysis** (Kramer, 1996) and it typically incorporates a finite-element or finite-difference mathematical model. In this type of analysis ground motion is incorporated in the form of an acceleration time history. Seismically induced permanent strains in each element of the finite element mesh are integrated to obtain the permanent deformation of the slope. The results of the analysis include a time history of the compressive and tensile stresses, natural frequencies, effects of damping, and slope displacements.

Because this type of analysis will only rarely be used for the types of projects affected by the Seismic Hazard Zone Maps, it will not be discussed further in these Guidelines.

Pseudo-Static Analysis

The ground-motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient "k". The selection of a seismic coefficient has relied heavily on engineering judgment and local code requirements because there is no simple method for determining an appropriate value. In California, many state and local agencies, on the basis of local experience, require the use of a seismic coefficient of 0.15, and a minimum computed pseudo-static factor of safety of 1.0 to 1.2 for analyses of natural, cut, and fill slopes. The evaluation should follow the lead agency practice guidelines for seismic coefficient and factor of safety values. If no local guidelines exist, the following discussion should assist in the estimation an appropriate seismic coefficient.

Cautionary Note: *The seismic coefficient "k" is **not** equivalent to the peak horizontal ground acceleration value, either probabilistic or deterministic; therefore PGA should **not** be used as a seismic coefficient in pseudo-static analyses. The use of PGA will usually result in overly conservative factors of safety (Seed, 1979; Chowdhury, 1978). Furthermore, the practice of reducing the PGA by a "repeatable acceleration" factor to obtain a pseudo-static coefficient has no basis in the scientific or engineering literature.*

Selection of a Seismic Coefficient

There have been a number of published articles that provide guidance in the selection of an appropriate seismic coefficient for pseudo-static analyses. Most can be regarded as being within a range of values enveloped by the recommendations of two publications, Seed (1979), and Hynes and Franklin (1984).

Seed's 1979 article (the 19th Rankine Lecture) summarizes the factors to be considered in evaluating dynamic stability of earth- and rock-fill embankments. After evaluating all of the available data on earthquake-induced deformations of embankment dams, Seed recommended some basic guidelines for making preliminary evaluations of embankments to ensure acceptable performance (i.e., permanent deformations which would not imperil the overall structural integrity of an embankment dam). These recommendations were: using a pseudo-static coefficient of 0.10 for magnitude 6½ earthquakes and 0.15 for magnitude 8¼ earthquakes, with an acceptable factor of safety of the order of 1.15. Seed believed that his guidelines would ensure that permanent ground deformations would be acceptably small. Seed also made extensive commentary on the choice of appropriate material strengths, and limited his recommendations to those embankments composed of materials that do not undergo severe strength loss due to seismic shaking with an expected crest acceleration of less than 0.75g.

Hynes and Franklin (1984) provided amplification factors to be used when considering the crest of an embankment in comparison to the input accelerations at the base, with the intention of identifying those embankments which could be expected to experience unacceptable deformations. They suggested using one-half the bedrock acceleration applied to the embankment crest with an acceptable factor of safety greater than 1.0, with a 20 percent reduction on material strengths. Hynes and Franklin limited the assessment to earthquakes of less than magnitude 8 with non-liquefiable materials comprising the embankment.

Although the two references discussed above were written specifically for application to earth embankments, they represent the best understanding of the range of appropriate seismic coefficients to use in slopes composed of other materials. Figure 1 presents a summary of the recommended values of "k" for the ranges of factor of safety and earthquake parameters presented in these two articles. Other suggested ranges have been added for comparison. Figure 1 is presented as a guide for selecting a seismic coefficient for a pseudo-static analysis in jurisdictions where pseudo-static coefficients have not been adopted by the lead agency.

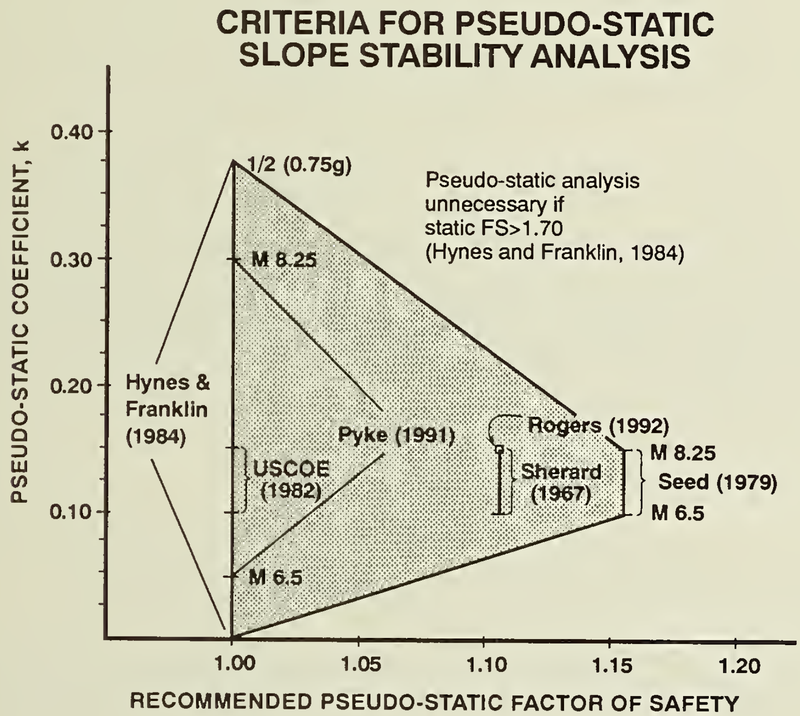

Figure 1. Approximate range of pseudo-static seismic coefficient "k" for anticipated factor of safety as proposed in the literature (references on the diagram).

Topographic Effects

Ashford and Sitar (1994) presented a method to analyze topographic amplification of site response on slopes. They specifically addressed the expected response of very steep slopes in weakly cemented rock. Amplification was found to increase with inclined seismic waves traveling into the slope crest. They found that the fundamental site period dominates the seismic response of any given slope. The relationship between wave length and slope height controls the degree of amplification. However, as the slopes decrease in steepness (i.e., less than 30 degrees), the slope-induced amplification becomes less and less important, and geologic contacts between dissimilar strata appear to exert more influence on observed failures.

Material Strengths

The pseudo-static analysis does not take into account any loss of material strength due to pore-pressure buildup along the anticipated slide surface due to earthquake loading (effective-stress conditions). For most investigations where the slopes are unsaturated or partially saturated, this assumption will be valid and the results of the analysis will tend to be conservative. If, however, the slopes being evaluated are saturated or are anticipated to be saturated, the loss of material strength during long-duration earthquake shaking may be expected and the analysis using total strength parameters may be more appropriate (see section on Effective-Stress vs. Total-Stress Conditions below).

Newmark Analysis

A Newmark analysis consists of three basic steps, as outlined below:

1. The first step is to perform a limit-equilibrium stability analysis to determine the location and shape of the critical slip surface (the slip surface with the lowest factor of safety), and the yield acceleration (k_y), defined as the acceleration required to bring the factor of safety to 1.0. Most computer-based slope stability programs include iterative routines for finding both of these parameters. If a computer program with these options is not available, the critical slip surface can be obtained through iterative trial-and-error, and the yield acceleration can be calculated from Newmark's relation

$$k_y = (FS - 1)g \sin a$$

   where FS is the static factor of safety, g is the acceleration due to gravity, and a is the angle from the horizontal that the center of mass of the landslide first moves.

2. The second step is to select an acceleration time history that represents the expected ground motions at the project site. The selection process typically involves estimating magnitude, source-to-site distance, and peak ground acceleration seismic parameters for the project site, and searching for existing strong-motion records that have parameters that closely match the estimated values. Methods for determining these site parameters and selecting a representative strong-motion record are outlined in Chapter 4. For Newmark analyses, Jibson (1993) recommended using: (1) Arias Intensity (Wilson, 1993; Wilson and Keefer, 1985), (2) magnitude and source distance, and (3) PGA and duration as criteria for selecting a suite of strong-motion records having characteristics of interest at a project site. Smith (1994a; 1994b) compiled a database of these characteristics for a large number of strong-motion records. Analysis of multiple records spanning a range of estimated shaking characteristics produces a range of calculated displacements, which provides a better basis for judgment of slope performance than one displacement calculated from a single record that may have unique idiosyncrasies. If the slopes to be analyzed are composed of soils or weakly cemented rock, and a strong-motion recording site with similar characteristics cannot be found, a seismic site response analysis should be performed. Houston and others (1987) described a method of using a one-dimensional wave propagation program (e.g., SHAKE91, Idriss and Sun, 1992; SHAKE, Schnabel and others, 1972) to find the average response at the slip surface prior to calculating displacements. As described in Chapter 4, sources for acquiring strong-motion records are provided in Appendix D.

3. The final step in a Newmark analysis is to calculate the cumulative displacements anticipated for the landslide under investigation. To do this, the design strong-motion record is integrated twice for those accelerations that exceed the yield acceleration, and the displacements are added to determine cumulative displacement. Computer software capable of calculating displacements from strong-motion records is available (Jibson, 1993; Houston and others, 1987) and can greatly simplify the analysis.

Jibson (1993) pointed out that, because Newmark's model assumes that landslides behave as rigid-plastic materials, the method might underestimate displacements for materials that lose shear strength as a function of strain, and overestimate displacements for soils that behave as viscoplastic materials. Due to the many assumptions that need to be made in the analysis, it is probably appropriate to consider calculations indicative only to within an order-of-magnitude of the actual displacements (e.g., centimeters, tens of centimeters, or meters). Considerable engineering judgment is required to establish "stability."

Effective-Stress vs. Total-Stress Conditions

In principle, a pseudo-static or Newmark analysis can be performed on either a total-stress or effective-stress basis. The geotechnical industry practice for "typical" developments has been to determine shear strength parameters from direct shear tests (effective-stress conditions) and assume that static and dynamic shear strengths are the same. For most investigations where the slopes are unsaturated or partially saturated, this assumption will be valid and the results of the analysis will tend to be conservative. However, for saturated slopes this assumption ignores the build-up of pore pressures due to dynamic loading, which can lower the shear resistance to failure and, in some cases, result in unconservative stability evaluations.

Seed (1966) presented an approach to a total-stress analysis for earth embankments that uses dynamic shear tests to derive a factor of safety that accounts for (a) initial conditions; (b) changes in stress and reorientation of principal stress; (c) decrease in strength due to cyclic loading conditions; and (d) decrease in strength due to undrained conditions during earthquake loading. This method is rigorous, and provides good estimates of the dynamic behavior of saturated materials but may be too costly for most projects.

A simpler approach to a total stress analysis would be to determine total-stress strength parameters from undrained triaxial shear tests and use those values in the stability analysis. Jibson and Keefer (1993) showed how to conduct such an analysis, and their results indicated that factors of safety and critical slip surfaces differed significantly from those generated from an effective stress analysis. The U.S. Army Corps of Engineers' practice is to use a composite shear strength envelope (based on a consolidated-drained test at low confining pressures and a consolidated-undrained test at high confining pressures) for permeable soils, and a consolidated-undrained strength envelope for soils with low permeability (Hynes and Franklin, 1984).

Makdisi and Seed (1978) have shown that substantial permanent displacements may be produced by cyclic loading of soils to stresses near the yield stress, while essentially elastic behavior is observed under many cycles of loading at 80 percent of the undrained strength. They recommend the use of 80 percent of the undrained strength for soils that exhibit small increases in pore pressure during cyclic loading, such as clayey soils, dry or partially saturated cohesionless soils, or very dense saturated cohesionless materials. This practice has been adopted by the U.S. Army

Corps of Engineers with an allowable pseudo-static factor of safety of 1.0 (Hynes and Franklin, 1984) and may be appropriate for many stability analyses in the absence of a more rigorous total stress analysis.

Evaluation of Potential Earthquake-Induced Landslide Hazards

The determination of dynamic slope stability (i.e., pseudo-static factors of safety or analytical displacements), and the acceptable parameters used in the analysis, should follow the standards defined by the lead agency. If no standards exist, the following general values may be used for defining the stability of slopes for static and dynamic loads.

Pseudo-Static Analysis

Slopes which have a pseudo-static factor of safety greater than 1.1 using an appropriate seismic coefficient can be considered stable. If the pseudo-static analysis results in a factor of safety lower than 1.1, the project engineer can either employ a Newmark analysis (or other displacement-type analysis method if acceptable to the lead agency) to determine the magnitude of slope displacements, or design appropriate mitigation measures.

Newmark Analysis

The Newmark analysis models a highly idealized and simplistic failure mechanism; thus, as discussed previously, the calculated displacements should be considered order-of-magnitude estimates of actual field behavior. Rather than being an accurate guide of observable landslide displacement in the field, Newmark displacements provide an index of probable seismic slope performance, and considerable judgment is required in evaluating seismic stability in terms of Newmark displacements. In some jurisdictions, less than 10 cm is considered stable; whereas, more than 30 cm is considered unstable. As a general guideline,

1. Newmark displacements of 0 to 10 cm are unlikely to correspond to serious landslide movement and damage.

2. In the 10 to 100 cm range, slope deformation may be sufficient to cause serious ground cracking or enough strength loss to result in continuing (post-seismic) failure. Determining whether displacements in this range can be accommodated safely requires good professional judgment that takes into account issues such as landslide geometry and material properties.

3. Calculated displacements greater than 100 cm are very likely to correspond to damaging landslide movement, and such slopes should be considered unstable.

Mitigation of Earthquake-Induced Landslide Hazards

Basic Considerations

For any existing or proposed slopes that are determined to be unstable, appropriate mitigation measures should be provided before the project is approved. The hazards these slopes present can be mitigated in one of three ways:

1. **Avoid the Failure Hazard:** Where the potential for failure is beyond the acceptable level and not preventable by practical means, as in mountainous terrain subject to massive planar slides or rock and debris avalanches, the hazard should be avoided. Developments should be built sufficiently far away from the threat that they will not be affected even if the slope does fail. Planned development areas on the slope or near its base should be avoided and relocated to areas where stabilization is feasible.

2. **Protect the Site from the Failure:** While it is not always possible to prevent slope failures occurring above a project site, it is sometimes possible to protect the site from the runout of failed slope materials. This is particularly true for sites located at or near the base of steep slopes which can receive large amounts of material from shallow disaggregated landslides or debris flows. Methods include catchment and/or protective structures such as basins, embankments, diversion or barrier walls, and fences. Diversion methods should only be employed where the diverted landslide materials will not affect other sites.

3. **Reduce the Hazard to an Acceptable Level:** Unstable slopes affecting a project can be rendered stable (that is, by increasing the factor of safety to > 1.5 for static and > 1.1 for dynamic loads) by eliminating the slope, removing the unstable soil and rock materials, or applying one or more appropriate slope stabilization methods (such as buttress fills, subdrains, soil nailing, crib walls, etc.). For deep-seated slope instability, strengthening the design of the structure (e.g., reinforced foundations) is generally not by itself an adequate mitigation measure.

The zones of required investigation for earthquake-induced landslides do not always include landslide or lateral spread run-out areas. Project sites that are outside of a zone of required investigation may be affected by ground-failure runout from adjacent or nearby slopes. Any proposed mitigation should address all recognized significant off-site hazards. If stabilization of source areas of potential off-site failures that could impact the project is not practical, it may be possible to achieve an acceptable level of risk by using one or more protective structures, as suggested below.

Stabilization Options

The stabilization method chosen depends largely on the type of instability which is anticipated at the project site. The two general techniques used to stabilize slopes are: (1) to reduce the driving force for failure, or (2) to increase the resisting force. These consist of different mechanisms, depending on the type of failures in question. The following list is presented to provide a range of stabilization options, but other options may be recommended provided analyses are presented to prove their validity.

Rock and Soil Falls

Principal failure mechanism is loss of cohesion or tensile strength of the near-surface material on a very steep slope.

Mitigation Strategies

1. **Reduce driving force** by reducing the steepness of the slope through grading, or by scaling off overhanging rock, diverting water from the slope face, etc.;

2. **Increase resisting force** by pinning individual blocks, covering the slope with mesh or net, or installing rock anchors or rock bolts on dense spacing; and/or,

3. **Protect the site from the failure** by constructing catchment structures such as basins, or protective structures such as walls and embankments.

Slides, Slumps, Block Glides

Principal failure mechanism is loss of shear strength, resulting in sliding of a soil or rock mass along a rupture surface within the slope.

Mitigation Strategies

1. **Reduce driving force,** by reducing the weight of the potential slide mass (cutting off the head of the slide, or totally removing the landslide), flattening the surface slope angle ('laying back' the slope face) through grading, preventing water infiltration by controlling surface drainage, or reducing the accumulation of subsurface water by installing subdrains; and/or,

2. **Increase resisting force,** by replacing slide debris and especially the rupture surface with compacted fill, installing shear keys or buttresses, dewatering the slide mass, pinning shallow slide masses with soil or rock anchors, reinforced caissons, or bolts, or constructing retaining structures at the edge of the slide.

Flows of Debris or Soil

Principal failure mechanism is fluidization of the soil mass, commonly by addition of water and possibly by earthquake shaking.

Mitigation Strategies

1. **Reduce driving force** by removing potential debris from site using grading or excavating procedures, or diverting water from debris so that it cannot mobilize, by means of surface drains and/or subsurface galleries or subdrains;

2. **Increase resisting force** by providing shear keys or buttresses, together with subsurface drainage; and/or,

Protect the site from the failure by diverting the flow away from project using diversion barriers or channels, or providing catchment structures to contain the landslide material.

CHAPTER 6

ANALYSIS AND MITIGATION OF LIQUEFACTION HAZARDS

Screening Investigations for Liquefaction Potential

The purpose of screening investigations for sites within zones of required investigation for liquefaction is to determine whether a given site has obvious indicators of a low potential for liquefaction failure (e.g., bedrock near the surface or deep ground water without perched water zones), or whether a more comprehensive field investigation is necessary to determine the potential for damaging ground displacements during earthquakes.

If a screening investigation can ***clearly*** demonstrate the absence of liquefaction hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement. If there is a reasonable expectation that liquefiable soils exist on the site and the engineering geologist and/or civil engineer can demonstrate that large lateral spread displacements (of more than 0.5 meter) are unlikely (e.g., Bartlett and Youd, 1995), the local agency may give them the option to forego the quantitative evaluation of liquefaction hazards and provide a structural mitigation for certain classes of structures. These mitigation methods are outlined in the mitigation section of this chapter. If the findings of the investigation fall outside these two options, then the more-comprehensive quantitative evaluation described below needs to be conducted.

Screening investigations for liquefaction hazards should address the following basic questions:

- **Are potentially liquefiable soil types present?**

  Given the highly variable nature of Holocene deposits that are likely to contain liquefiable materials, most sites will require borings to determine whether liquefiable materials underlie the project site. Borings used to define subsurface soil properties for other purposes (e.g., foundation investigations, environmental or groundwater studies) may provide valuable subsurface geologic and/or geotechnical information.

  The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. However, cohesive soils with: (a) a clay content (percent finer than 0.005 mm) less than 15 percent, (b) a liquid limit less than 35 percent, and (c) a moisture content of the in-place soil that is greater than 0.9 times the liquid limit (i.e., sensitive clays), are vulnerable to significant strength loss under relatively minor strains (Seed and others, 1983). Although not

classically defined as "liquefaction" and so not addressed by these Guidelines, these soils represent an additional seismic hazard that, if present, should be addressed.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction. Most gravelly soils drain relatively well, but when: (a) their voids are filled with finer particles, or (b) they are surrounded by less pervious soils, drainage can be impeded and they may be vulnerable to cyclic pore pressure generation and liquefaction. Gravelly geologic units tend to be deposited in a more-turbulent depositional environment than sands or silts, tend to be fairly dense, and so generally resist liquefaction. Accordingly, conservative "preliminary" methods may often suffice for evaluation of their liquefaction potential. For example, gravelly deposits which can be shown to be pre-Holocene in age (older than about 11,000 years) are generally not considered susceptible to liquefaction.

- **If present, are the potentially liquefiable soils saturated or might they become saturated?**

In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, liquefaction hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths. If it can be demonstrated that any potentially liquefiable materials present at a site: (a) are currently unsaturated (e.g., are above the water table), (b) have not previously been saturated (e.g., are above the historic-high water table), and (c) are highly unlikely to become saturated (given foreseeable changes in the hydrologic regime), then such soils generally do not constitute a liquefaction hazard that would require mitigation. Note that project development, changes in local or regional water management patterns, or both, can significantly raise the water table or create zones of perched water. Extrapolating water table elevations from adjacent sites does not, by itself, demonstrate the absence of liquefaction hazards, except in those unusual cases where a combination of uniformity of local geology and very low regional water tables permits very conservative assessment of water table depths. Screening investigations should also address the possibility of local "perched" water tables, the raising of water levels by septic systems, or the presence of locally saturated soil units at a proposed project site.

- **Is the geometry of potentially liquefiable deposits such that they pose significant risks requiring further investigation, or might they be mitigated by relatively inexpensive foundation strengthening?**

Relatively thin seams of liquefiable soils (on the order of only a few centimeters thick), if laterally continuous over sufficient area, can represent potentially hazardous planes of weakness and sliding, and may thus pose a hazard with respect to lateral spreading and related ground displacements. Thus, the screening investigation should identify nearby free faces (cut slopes, streambanks, and shoreline areas), whether on or off-site, to determine whether lateral spreading and related ground displacements might pose a hazard to the project. If such features are found, the quantitative evaluation of liquefaction usually will be warranted because of potential life-safety concerns.

Even when it is not possible to demonstrate the absence of potentially liquefiable soils or prove that such soils are not and will not become saturated, it may be possible to demonstrate

that any potential liquefaction hazards can be adequately mitigated through a simple strengthening of the foundation of the structure, as described in the mitigation section of this chapter, or other appropriate methods.

- **Are in-situ soil densities sufficiently high to preclude liquefaction?**

If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location which might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate than those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. Direct in-situ relative density measurements, such as the ASTM D 1586-92 (Standard Penetration Test [SPT]) or ASTM D3441-94 (Cone Penetration Test [CPT]).

2. Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). Current groundwater level data, including perched water tables, may be obtained from permanent wells, driller's logs and exploratory borings. Historical groundwater data can be found in reports by various government agencies, although such reports often provide information only on water from production zones and ignore shallower water.

3. Non-standard penetration test data. It should be noted that correlation of non-standard penetration test results (e.g., sampler size, hammer weight/drop, hollow stem auger) with SPT resistance is very approximate, and so requires very conservative interpretation, unless direct SPT and non-standard test comparisons are made at the site and in the materials of interest.

4. Geophysical measurements of shear-wave velocities.

5. "Threshold strain" techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards is very low.

Quantitative Evaluation of Liquefaction Resistance

Liquefaction investigations are best performed as part of a comprehensive investigation. These Guidelines are to promote uniform evaluation of the resistance of soil to liquefaction.

Detailed Field Investigation

Engineering Geologic Investigations

Engineering geologic investigations should determine:

1. The presence, texture (e.g., grain size), and distribution (including depth) of unconsolidated deposits;

2. The age of unconsolidated deposits, especially for Quaternary Period units (both Pleistocene and Holocene Epochs);

3. Zones of flooding or historic liquefaction; and,

4. The groundwater level to be used in the liquefaction analysis, based on data from well logs, boreholes, monitoring wells, geophysical investigations, or available maps. Generally, the historic high groundwater level should be used unless other information indicates a higher or lower level is appropriate.

The engineering geologic investigations should reflect relative age, soil classification, three-dimensional distribution and general nature of exposures of earth materials within the area. Surficial deposits should be described as to general characteristics (including environment of deposition) and their relationship to present topography and drainage. It may be necessary to extend the mapping into adjacent areas. Geologic cross sections should be constrained by boreholes and/or trenches when available.

Geotechnical Field Investigation

The vast majority of liquefaction hazards are associated with sandy and/or silty soils. For such soil types, there are at present two approaches available for quantitative evaluation of the soil's resistance to liquefaction. These are: (1) correlation and analyses based on in-situ Standard Penetration Test (SPT) (ASTM D1586-92) data, and (2) correlation and analyses based on in-situ Cone Penetration Test (CPT) (ASTM D3441-94) data. Both of these methods have some relative advantages (see Table 4). Either of these methods can suffice by itself for some site conditions, but there is also considerable advantage to using them jointly.

Seed and others (1985) provide guidelines for performing "standardized" SPT, and also provide correlations for conversion of penetration resistance obtained using most of the common alternate combinations of equipment and procedures in order to develop equivalent "standardized" penetration resistance values— $(N_1)_{60}$. These "standardized" penetration resistance values can then be used as a basis for evaluating liquefaction resistance.

Table 4. Comparative advantages of SPT and CPT methods.

SPT ADVANTAGES	CPT ADVANTAGES
1. Retrieves a sample. This permits identification of soil type ***with certainty***, and permits evaluation of fines content (which influences liquefaction resistance). Note that CPT provides poor resolution with respect to soil classification, and so usually requires some complementary borings with samples to more reliably define soil types and stratigraphy.	1. Provides ***continuous*** penetration resistance data, as opposed to averaged data over discrete increments (as with SPT), and so is less likely to "miss" thin layers and seams of liquefiable material.
2. Liquefaction resistance correlation is based primarily on field case histories, and the vast majority of the field case history database is for in-situ SPT data.	2. Faster and less expensive than SPT, as no borehole is required.

Cone penetration test (CPT) tip resistance (q_c) may also be used as a basis for evaluation of liquefaction resistance, by either (a) direct empirical comparison between q_c data and case histories of seismic performance (Olsen, 1988), or (b) conversion of q_c-values to "equivalent" $(N_1)_{60}$-values and use of correlations between $(N_1)_{60}$ data and case histories of seismic performance. At present, Method (b)— conversion of q_c to equivalent $(N_1)_{60}$— is preferred because the field case history data base for SPT is well-developed compared to CPT correlations. A number of suitable correlations between q_c and $(N_1)_{60}$ are available (e.g., Robertson and Campanella, 1985; Seed and De Alba, 1986). These types of conversion correlations depend to some extent on knowledge of soil characteristics (e.g., soil type, mean particle size (D_{50}), fines content). When the needed soil characteristics are either unknown or poorly defined, then it should be assumed that the ratio

$$\frac{q_c\left(kg / cm^2\right)}{N(blows / ft)}$$

is approximately equal to 5 for conversion from q_c to "equivalent" N-values.

Geotechnical Laboratory Testing

The use of laboratory testing (e.g., cyclic triaxial, cyclic simple shear, cyclic torsional tests) on "undisturbed" soil samples as the sole basis for the evaluation of in-situ liquefaction resistance is not recommended, as unavoidable sample disturbance and/or sample densification during reconsolidation prior to undrained cyclic shearing causes a largely unpredictable, and typically unconservative, bias to such test results. Laboratory testing is recommended for determining

grain-size distribution (including mean grain size D_{50}, effective grain size D_{10}, and percent passing #200 sieve), unit weights, moisture contents, void ratios, and relative density.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction (Evans and Fragasy, 1995, Evans and Zhou, 1995). Most gravelly soils drain relatively well, but when their voids are filled with finer particles, or they are surrounded (or "capped") by less pervious soils, drainage can be impeded and they may be vulnerable to liquefaction. Gravelly soils tend to be deposited in a more turbulent environment than sands or silts, and are fairly dense, and so are generally resistant to liquefaction. Accordingly, conservative "preliminary evaluation" methods (e.g., geologic assessments and/or shear-wave velocity measurements) often suffice for evaluation of their liquefaction potential. When preliminary evaluation does not suffice, more accurate quantitative methods must be used. Unfortunately, neither SPT nor CPT provides reliable penetration resistance data in soils with high gravel content, as the large particles impede these small-diameter penetrometers. At present, the best available technique for quantitative evaluation of the liquefaction resistance of coarse, gravelly soils involves correlations and analyses based on in-situ penetration resistance measurements using the very large-scale Becker-type Hammer system (Harder, 1988).

Evaluation of Potential Liquefaction Hazards

The factor of safety for liquefaction resistance has been defined as:

$$\text{Factor of Safety} = \frac{CSR_{liq}}{CSR_{eq}}$$

where CSR_{eq} is the cyclic stress ratio generated by the anticipated earthquake ground motions at the site, and CSR_{liq} is the cyclic stress ratio required to generate liquefaction (Seed and Idriss, 1982). For the purposes of evaluating the results of a quantitative assessment of liquefaction potential at a site, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk. This factor of safety assumes that high-quality, site-specific penetration resistance and geotechnical laboratory data were collected, and that ground-motion data from DMG (Petersen and others, 1996) were used in the analyses. If lower factors of safety are calculated for some soil zones, then an evaluation of the level (or severity) of the hazard associated with potential liquefaction of these soils should be made.

Such hazard assessment requires considerable engineering judgment. The following is, therefore, only a guide. The assessment of hazard associated with potential liquefaction of soil deposits at a site must consider two basic types of hazard:

1. Translational site instability (sliding, edge failure, lateral spreading, flow failure, etc.) that potentially may affect all or large portions of the site; and

2. More localized hazard at and immediately adjacent to the structures and/or facilities of concern (e.g., bearing failure, settlement, localized lateral movements).

As Bartlett and Youd (1995) have stated: "Two general questions must be answered when evaluating the liquefaction hazards for a given site:

(1) 'Are the sediments susceptible to liquefaction?'; and

(2) 'If liquefaction does occur, what will be the ensuing amount of ground deformation?'"

Lateral Spreading and Site Displacement Hazards

Lateral spreading on gently sloping ground generally is the most pervasive and damaging type of liquefaction failure (Bartlett and Youd, 1995). Assessment of the potential for lateral spreading and other large site displacement hazards may involve the need to determine the residual undrained strengths of potentially liquefiable soils. If required, this should be done using in-situ SPT or CPT test data (e.g., Seed and Harder, 1990). The use of laboratory testing for this purpose is not recommended, as a number of factors (e.g., sample disturbance, sample densification during reconsolidation prior to undrained shearing, and void ratio redistribution) render laboratory testing a potentially unreliable, and, therefore, unconservative basis for assessment of in-situ residual undrained strengths. Assessment of residual strengths of silty or clayey soils may, however, be based on laboratory testing of "undisturbed" samples.

Assessment of potential lateral spread hazards must consider dynamic loading as a potential "driving" force, in addition to gravitational forces. It should again be noted that relatively thin seams of liquefiable material, if fairly continuous over large lateral areas, may serve as significant planes of weakness for translational movements. If prevention of translation or lateral spreading is ascribed to structures providing "edge containment," then the ability of these structures (e.g., berms, dikes, sea walls) to resist failure must also be assessed. Special care should be taken in assessing the containment capabilities of structures prone to potentially "brittle" modes of failure (e.g., brittle walls which may break, tiebacks which may fail in tension). If a hazard associated with potentially large translational movements is found to exist, then either: (a) suitable recommendations for mitigation of this hazard should be developed, or (b) the proposed "project" should be discontinued.

When suitably sound lateral containment is demonstrated to prevent potential sliding on liquefied layers, then potentially liquefiable zones of finite thickness occurring at depth may be deemed to pose no significant risk beyond the previously defined minimum acceptable level of risk. Suitable criteria upon which to base such an assessment include those proposed by Ishihara (1985, Figure 88; 1996, Chapter 16).

For information on empirical models that might be appropriate to use in these analyses, see Bartlett and Youd (1995).

Localized Liquefaction Hazards

If it can be shown that no significant risk of large translational movements exists, or if suitable mitigation measures can be developed that address such risks, then studies should proceed to consideration of five general types of more localized potential hazards, including:

1. ***Potential foundation bearing failure, or large foundation settlements due to ground softening and near-failure in bearing.*** To form a basis for concluding that no hazard exists, a high factor of safety (FS > 1.5) should be based on a realistic appraisal of the minimum soil strengths likely to be mobilized to resist bearing failure (including residual undrained strengths of soils considered likely to liquefy or to suffer significant strength loss due to cyclic pore pressure generation). If such hazard does exist, then appropriate recommendations for mitigation of this hazard should be developed.

2. ***Potential structural and/or site settlements.*** Settlements for saturated and unsaturated clean sands can be estimated using simplified empirical procedures (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshmine, 1992). These procedures, developed for relatively clean, sandy soils, have been found to provide reasonably reliable settlement estimates for sites not prone to significant lateral spreading.

   Any prediction of liquefaction-related settlements is necessarily approximate, and related hazard assessment and/or development of recommendations for mitigation of such hazard should, accordingly, be performed with suitable conservatism. Similarly, it is very difficult to reliably estimate the amount of localized ***differential*** settlement likely to occur as part of the overall predicted settlement: localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made.

3. ***Localized lateral displacement; "lateral spreading" and/or lateral compression.*** Methods for prediction of lateral ground displacements due to liquefaction-related ground softening are not yet well supported by data from case histories of field performance. As such case history data are now being developed, significant advances in the reliability and utility of techniques for prediction of lateral displacements may be expected over the next few years. Finite element models represent the most sophisticated method currently in use for calculating permanent displacements due to liquefaction lateral spreading. Like the dynamic analysis for landslide displacements, this method evaluates time histories of the stresses and strains for a strong-motion time history. This method is a state-of-the-art approach to liquefaction hazards and will likely take time to become the state-of-the-practice.

   Consultants performing liquefaction hazard assessment should do their best to keep abreast of such developments. At present, lateral ground displacement magnitudes can be predicted with reasonable accuracy and reliability only for cases wherein such displacements are likely to be "small" (e.g., on the order of 15 cm or less). Larger displacements may be predicted with an accuracy of $\pm$ one meter or more; this level of accuracy may suffice for design of some structures (e.g., earth and rock-fill dams), but does not represent a sufficiently refined level of accuracy as to be of use for design of foundations for most types of structures.

   It may be possible to demonstrate that localized lateral displacements will be 0.5 meter or less based on: (a) evaluation of soil stratigraphy, residual undrained strengths, and duration and severity of seismic loading, or (b) simplified empirical methods. Bartlett and Youd's (1995) empirical procedure uses an existing field case history database of lateral

spread occurrences. Other empirical methods or more complex analyses, may yield somewhat different results but should be allowed if the methods are documented and the results justified. When likely maximum lateral displacements can be shown to be less than 0.5 meter (e.g., Bartlett and Youd, 1995), it may be possible to design foundations with sufficient strength to withstand the expected movements without complete failure. In all other cases, more extensive recommendations are needed for mitigation of the hazard associated with potential lateral displacements.

4. ***Floatation of light structures with basements, or underground storage structures.*** Light structures which extend below the groundwater table and contain large void spaces may "float" or rise out of the ground during, or soon after an earthquake. Structures that are designed for shallow groundwater conditions typically rely on elements, such as cantilevered walls or tie-downs, that resist the buoyant or uplift forces produced by the water. If the material surrounding these elements liquefies, the resisting forces can be significantly reduced and the entire structure may be lifted out of the ground.

5. ***Hazards to Lifelines.*** To date, most liquefaction hazard investigations have focused on assessing the risks to commercial buildings, homes, and other occupied structures. However, liquefaction also poses problems for streets and lifelines— problems that may, in turn, jeopardize lives and property. For example, liquefaction locally caused natural gas pipelines to break and catch fire during the Northridge earthquake, and liquefaction-caused water line breakage greatly hampered firefighters in San Francisco following the 1906 earthquake. Thus, although lifelines are not explicitly mentioned in the Seismic Hazards Mapping Act, cities and counties may wish to require investigation and mitigation of potential liquefaction-caused damage to lifelines.

Mitigation of Liquefaction Hazards

The hazard assessment required for project sites within zones of required investigation should (a) demonstrate that liquefaction at a proposed project site poses a sufficiently low hazard as to satisfy the defined acceptable level of risk criteria, or (b) result in implementation of suitable mitigation recommendations to effectively reduce the hazard to acceptable levels (CCR Title 14, Section 3721). Mitigation should provide suitable levels of protection with regard to the two general types of liquefaction hazards previously discussed (1) potential large lateral spread failures, and (2) more localized problems including potential bearing failure, settlements, and lateral displacements.

Potentially suitable methods for mitigation of lateral spread hazards may include the following:

1. Edge containment structures (e.g., berms, dikes, sea walls, retaining structures, compacted soil zones);

2. Removal or treatment of liquefiable soils to reduce liquefaction potential;

3. Modification of site geometry to reduce the risk of translational site instability; and/or

4. Drainage to lower the groundwater table below the level of the liquefiable soils.

Mitigation techniques may be applied individually or in combination. Mitchell and others (1995) summarize the performance of some mitigation techniques for past earthquakes. Hryciw (1995) includes several articles with additional information about the success of specific soil improvement techniques.

Once problems related to potentially large lateral spread failures have been resolved, the remaining "localized" potential hazards should be addressed and resolved. Suitable mitigation alternatives may include one or more of the following:

1. Excavation and removal or recompaction of potentially liquefiable soils;

2. In-situ ground densification (e.g., compaction with vibratory probes, dynamic consolidation, compaction piles, blasting densification, compaction grouting);

3. Other types of ground improvement (e.g., permeation grouting, columnar jet grouting, deep mixing, gravel drains or other drains, surcharge pre-loading, structural fills, dewatering);

4. Deep foundations (e.g., piles, piers), that have been designed to accommodate liquefaction effects;

5. Reinforced shallow foundations (e.g., grade beams, combined footings, reinforced or post-tensioned slabs, rigid raft foundations); and

6. Design of the proposed structures or facilities to withstand predicted ground softening and/or predicted vertical and lateral ground displacements to an acceptable level of risk.

The scope and type(s) of mitigation required depend on the site conditions present and the nature of the proposed project. Individual mitigation techniques may be used, but the most appropriate solution may involve using them in combination.

In general, only removal and/or densification of potentially liquefiable soils, or drainage of groundwater can ***fully*** eliminate all liquefaction hazards. In many cases, other methods may achieve the desired acceptable level of risk. For example, in areas where liquefaction may potentially cause displacements of one-third meter or less, design of the foundation to withstand displacements of one-half meter can significantly reduce future damage from liquefaction. The Northridge earthquake caused liquefaction in a number of locations. Insurers reported that losses equal to two-thirds of the value of damaged structures were not uncommon— structures that took many months, if not years, to again make habitable. Youd (personal communication, 1996) and other engineers indicate that by adding adequate reinforcing steel to properly designed concrete slabs or grade beams to resist fracture during ground displacement (very inexpensive for a single-family dwelling), 80 percent or more of this damage would have been avoided and repairs (patching, re-leveling of homes, etc.) would have been expedited. Such improved foundations will also reduce damage from expansive soils, settling, minor landslide movement, and similar ground-related problems (Federal Emergency Management Agency, in press). Based on these conclusions, the Liquefaction Working Group strongly recommends that, if the consultant determines that the project site will be affected by small lateral spreading, lead agencies should consider waiving detailed site investigations in lieu of foundation and structure designs that safely

withstand up to two times the estimated deformations without fracturing the foundation. In the Liquefaction Working Group's opinion, the money required for detailed site investigations in areas not subject to lateral spread displacement would be better spent on mitigation than on investigation. This mitigation measure should provide adequate protection to the structure but will leave buried utilities unprotected and subject to damage, particularly at connections to the improved structures. In zones of required investigation for liquefaction, developers and utility companies should use types of pipe and flexible connections that are resistant to earthquake damage, thereby increasing the likelihood that the utilities will be functional after an earthquake (Federal Emergency Management Agency, in press).

Development of appropriate recommendations for mitigation of liquefaction hazards requires considerable judgment, as does the review and evaluation of such recommendations. Accordingly, the importance of the lead agency technical reviewer is emphasized. Technical reviewers are reminded to consider that the intent of the State's Seismic Hazard Zone program is to provide an adequate minimum level of protection for projects in the zone of required investigation, based on the acceptable level of risk. Owners/developers are, however, also hereby encouraged to implement a higher level of mitigation, in order to protect their investment and/or to minimize their potential future exposure and that of future occupants or users of the project structures or facilities.

CHAPTER 7

GUIDELINES FOR REVIEWING SITE-INVESTIGATION REPORTS

The purpose of this chapter is to provide general guidance to regulatory agencies that have approval authority over projects and to engineering geologists and civil engineers who review reports of seismic hazard investigations. These Guidelines recognize that effective mitigation ultimately depends on the professional judgment and expertise of the developer's engineering geologist and/or civil engineer in concert with the lead agency's engineering geologist and/or civil engineer.

The required technical review is a critical part of the evaluation process of approving a project. The reviewer ensures compliance with existing laws, regulations, ordinances, codes, policies, standards, and good practice, helping to assure that significant geologic factors (hazards and geologic processes) are properly considered, and potential problems are mitigated prior to project development. Under the Seismic Hazards Mapping Act, the reviewer is responsible for determining that each seismic hazard site investigation, and the resulting report, reasonably address the geologic and soil conditions that exist at a given site. The reviewer acts on behalf of a governing agency— city, county, regional, state, or federal— not only to protect the government's interest but also to protect the interest of the community at large. Examples of the review process in a state agency are described by Stewart and others (1976). Review at the local level has been discussed by Leighton (1975), Hart and Williams (1978), Berkland (1992), and Larson (1992). Grading codes, inspections, and the review process are discussed in detail by Scullin (1983).

The Reviewer

Qualifications

CCR Title 14, Section 3724(c) states that the reviewer must be a licensed engineering geologist and/or civil engineer having competence in the field of seismic hazard evaluation and mitigation. California's Business and Professions Code limits the practice of geology and engineering to licensed geologists and engineers, respectively, thereby requiring that reviewers be licensed, or directly supervised by someone who is licensed, by the appropriate State board. Local and regional agencies may have additional requirements. Nothing in these Guidelines is intended to sanction or authorize the review of engineering geology reports by engineers or civil engineering reports by geologists.

The reviewer should be familiar with the investigative methods employed and the techniques available to these professions (see Chapters 3 through 6). The opinions and comments made by the reviewer should be competent, prudent, objective, consistent, unbiased, pragmatic, and reasonable. The reviewer should be professional and ethical. The reviewer should have a clear understanding of the criteria for approving and not approving reports. Reviews should be based on logical, defensible criteria.

Reviewers must recognize their limitations. They should be willing to ask for the opinions of others more qualified in specialty fields.

If there is clear evidence of incompetence or misrepresentation in a report, this fact should be reported to the reviewing agency or licensing board. California Civil Code Section 47 provides an immunity for statements made "in the initiation or course of any other proceedings authorized by law." Courts have interpreted this section as providing immunity to letters of complaint written to provide a public agency or board, including licensing boards, with information that the public board or agency may want to investigate (see *King v. Borges,* 28 Cal. App. 3d 27 [1972]; and *Brody v. Montalbano,* 87 Cal. App 3d 725 [1978]). Clearly, reviewers need to have the support of their agency in order to carry out these duties.

The primary purpose of the review procedure should always be kept in mind: to determine compliance with the regulations, codes, and ordinances that pertain to the development. The reviewer should demand that minimum standards are met. The mark of a good reviewer is the ability to sort out the important from the insignificant, to list appropriate requirements for compliance, and to assist the applicant and their consultants in meeting the regulations without doing the consultant's job.

Conflict of Interest

In cases where reviewers also perform geologic or engineering investigations, they should ***never*** be placed in the position of reviewing their own report, or that of their own agency or company.

Reviewing Reports

The Report

A report that is incomplete or poorly written should be ***not*** approved. The report should demonstrate that the project complies with applicable regulations, codes, and ordinances, or local functional equivalents, in order to be approved.

The reviewer performs four principal functions in the technical review:

1. Identify any known potential hazards and impacts that are not addressed in the consultant's report. The reviewer should require investigation of the potential hazards and impacts;

2. Determine that the report contains sufficient data to support and is consistent with the stated conclusions;

3. Determine that the conclusions identify the potential impact of known and reasonable anticipated geologic processes and site conditions during the lifespan of the project; and,

4. Determine that the recommendations are consistent with the conclusions and can reasonably be expected to mitigate those anticipated earthquake-related problems that could have a significant impact on the proposed development. The included recommendations also should address the need for additional geologic and engineering investigations (including any site inspections to be made as site remediation proceeds).

Report Guidelines and Standards

Investigators may save a great deal of time (and the client's money), and possibly misunderstandings, if they contact the reviewing geologist or engineer at the initiation of the investigation. Reviewers typically are familiar with the local geology and sources of information and may be able to provide additional guidance regarding their agency's expectations and review practices. Guidelines for geologic or geotechnical reports have been prepared by a number of agencies and are available to assist reviewers in their evaluation of reports (for example, DMG Notes 42, 44, 48, and 49). Distribution of copies of written policies and guidelines adopted by the agency, usually alerts the applicants and consultants about procedures, report formats, and levels of investigative detail that will expedite review and approval of the project.

If a reviewer determines that a report is not in compliance with the appropriate requirements, this fact should be stated in the written record. After the reviewer is satisfied that the investigation and resulting conclusions and recommendations are reasonable and meet local requirements, approval of the project should be recommended to the reviewing agency.

Review of Submitted Reports

The review of submitted reports constitutes professional practice and should be conducted as such. The reviewer should study the available data and site conditions in order to determine whether the report is in compliance with local requirements. A field reconnaissance of the site should be conducted, preferably after the review of available stereoscopic aerial photographs, geologic maps, and reports on nearby developments.

For each report reviewed, a clear, concise, and logical written record should be developed. This review record may be as long or short as is necessary, depending upon the complexity of the project, the geology, the engineering analysis, and the quality and completeness of the reports submitted. At a minimum, the record should:

1. Identify the project, pertinent permits, applicant, consultants, reports and plans reviewed;

2. Include a clear statement of the requirements to be met by the parties involved, data required, and the plan, phase, project, or report being approved or denied;

3. Contain summaries of the reviewer's field observations, associated literature and air photo review, and oral communications with the applicant and the consultant; and,

4. Contain copies of any pertinent written correspondence.

5. The reviewer's name and license number(s), with any associated expiration dates.

The report, plans, and review record should be kept in perpetuity to document that compliance with local requirements was achieved and for reference during future development, remodeling, or rebuilding. Such records also can be a valuable resource for land-use planning and real-estate disclosure.

Report Filing Requirements

PRC Section 2697 requires cities and counties to submit one copy of each approved site-investigation report, including mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of report approval. Section 2697 also requires that if a project's approval is not in accordance with the policies and criteria of the State Mining and Geology Board (CCR Title 14, Chapter 2, Division 8, Article 10), the city or county must explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval. Reports should be sent to:

California Department of Conservation
Division of Mines and Geology
Attn: Seismic Hazard Reports
801 K Street, MS 12-31
Sacramento, CA 95814-3531

Waivers

PRC Section 2697 and CCR Title 14, Section 3725 outline the process under which lead agencies may determine that information from studies conducted on sites in the immediate vicinity may be used to waive the site-investigation report requirement. CCR Title 14, Section 3725 indicates that when a lead agency determines that "geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required," it may grant a waiver. CCR Title 14, Section 3725 also requires that such a finding be based on a report presenting evaluations of sites in the immediate vicinity having similar geologic and geotechnical characteristics. Further, Section 3725 stipulates that lead agencies must review waiver requests in the same manner as it reviews site-investigation reports. Thus waiver requests must be reviewed by a licensed engineering geologist and/or civil engineer having competence in the field of seismic hazard evaluation and mitigation. Generally, in addition to the findings of the reports that are presented in support of the waiver request, reviewers should consider:

1. The proximity of the project site to sites previously evaluated;

2. Whether the project sites previously evaluated adequately "surround" the project site to preclude the presence of stream channel deposits, historically higher water table, stream channels and other types of free faces that may present an opportunity for lateral spread failures; and,

3. Whether the supporting reports do, in fact, conclude that no hazard exists.

Waiver Filing Requirements

CCR Title 14, Section 3725 provides that "All such waivers shall be recorded with the county recorder and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver." These materials should be sent to:

> California Department of Conservation
> Division of Mines and Geology
> Attn: Seismic Hazard Reports
> 801 K Street, MS 12-31
> Sacramento, CA 95814-3531

Appeals

In cases where the reviewer is not able to approve a site-investigation report, or can accept it only on a conditional basis, the developer may wish to appeal the review decision. However, every effort should be made to resolve problems informally prior to making a formal appeal. Appeal procedures are often specified by a city or county ordinance or similar instrument. An appeal may be handled through existing legal procedures, such as a hearing by a County Board of Supervisors, a City Council, or a specially appointed Technical Appeals and Review Panel. Several administrators note that the Technical Appeals and Review Panel, comprised of geoscientists, engineers, and other appropriate professionals, benefits decision makers by providing additional technical expertise for especially complex and/or controversial cases. Adequate notice should be given to allow time for both sides to prepare their cases. After an appropriate hearing, the appeals decision should be made promptly and in writing as part of the permanent record.

Another way to remedy conflicts between the investigator and the reviewer is by means of a third party review. Such a review can take different paths ranging from the review of existing reports to in-depth field investigations. Third party reviews are usually done by consultants who are not normally associated with the reviewing/permitting agency.

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APPENDIX A

SEISMIC HAZARDS MAPPING ACT

PUBLIC RESOURCES CODE

Division 2. Geology, Mines, and Mining

CHAPTER 7.8. SEISMIC HAZARDS MAPPING

2690. This chapter shall be known and may be cited as the Seismic Hazards Mapping Act.

2691. The Legislature finds and declares all of the following:

(a) The effects of strong ground shaking, liquefaction, landslides, or other ground failure account for approximately 95 percent of economic losses caused by an earthquake.

(b) Areas subject to these processes during an earthquake have not been identified or mapped statewide, despite the fact that scientific techniques are available to do so.

(c) It is necessary to identify and map seismic hazard zones in order for cities and counties to adequately prepare the safety element of their general plans and to encourage land use management policies and regulations to reduce and mitigate those hazards to protect public health and safety.

2692. (a) It is the intent of the Legislature to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in fulfilling their responsibilities for protecting the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

(b) It is further the intent of the Legislature that maps and accompanying information provided pursuant to this chapter be made available to local governments for planning and development purposes.

(c) It is further the intent of the Legislature that the Division of Mines and Geology, in implementing this chapter, shall, to the extent possible, coordinate its activities with, and use existing information generated from, the earthquake fault zones mapping program pursuant to Chapter 7.5 (commencing with Section 2621), the landslide hazard identification program pursuant to Chapter 7.7 (commencing with Section 2670), and the inundation maps prepared pursuant to Section 8589.5 of the Government Code.

2692.1. The State Geologist may include in maps compiled pursuant to this chapter information on the potential effects of tsunami and seiche when information becomes available from other sources and the State Geologist determines the information is appropriate for use by local government. The State Geologist shall not be required to provide this information unless additional funding is provided both to make the determination and to distribute the tsunami and seiche information.

2693. As used in this chapter:

(a) "City" and "county" includes the City and County of San Francisco.

(b) "Geotechnical report" means a report prepared by a certified engineering geologist or a civil engineer practicing within the area of his or her competence, which identifies seismic hazards and recommends mitigation measures to reduce the risk of seismic hazard to acceptable levels.

(c) "Mitigation" means those measures that are consistent with established practice and that will reduce seismic risk to acceptable levels.

(d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621), except as follows:

(1) A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.

(2) "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.

(e) "Commission" means the Seismic Safety Commission.

(f) "Board" means the State Mining and Geology Board.

2694. (a) A person who is acting as an agent for a seller of real property which is located within a seismic hazard zone, as designated under this chapter, or the seller if he or she is acting without an agent, shall disclose to any prospective purchaser the fact that the property is located within a seismic hazard zone, if the maps prepared pursuant to this chapter or the information contained in the maps are reasonably available.

(b) For the purposes of this section, in all transactions subject to Section 1102 of the Civil Code, disclosure shall be provided by one of the following means:

(1) The real estate transfer disclosure statement set out in Section 1102.6 of the Civil Code.

(2) The local option real estate transfer disclosure statement set out in subdivision (a) of Section 1102.6 of the Civil Code.

(3) The real estate contract and receipt for deposit.

(c) For purposes of this section:

(1) "Reasonably available" means that for any county that includes areas covered by seismic hazard maps, a notice is posted at the offices of the county recorder, county assessor, and county planning commission. The notice shall identify the location of the maps and the effective date of the notice, which shall not be 10 days beyond the date the county received the maps from the State Geologist. The notice may also be posted at any other location determined by the county to be necessary to achieve adequate distribution.

(2) "Real estate contract and receipt for deposit" means the document containing the offer to sell or purchase real property, that when accepted becomes a binding contract, and that serves as an acknowledgment of a deposit if one is received.

(d) For purposes of the disclosures required by this section, the following persons shall not be deemed agents of the transferor:

(1) Persons specified in Section 1102.11 of the Civil Code.

(2) Persons acting under a power of sale regulated by Section 2924 of the Civil Code.

(e) For purposes of this section, Section 1102. 13 of the Civil Code applies.

2695. (a) On or before January 1, 1992, the board, in consultation with the director and the commission, shall develop all of the following:

(1) Guidelines for the preparation of maps of seismic hazard zones in the state.

(2) Priorities for mapping of seismic hazard zones. In setting priorities, the board shall take into account the following factors:

(A) The population affected by the seismic hazard in the event of an earthquake.

(B) The probability that the seismic hazard would threaten public health and safety in the event of an earthquake.

(C) The willingness of lead agencies and other public agencies to share the cost of mapping within their jurisdiction.

(D) The availability of existing information.

(3) Policies and criteria regarding the responsibilities of cities, counties, and state agencies pursuant to this chapter. The policies and criteria shall address, but not be limited to, the following:

(A) Criteria for approval of a project within a seismic hazard zone, including mitigation measures.

(B) The contents of the geotechnical report.

(C) Evaluation of the geotechnical report by the lead agency.

(4) Guidelines for evaluating seismic hazards and recommending mitigation measures.

(5) Any necessary procedures, including, but not limited to, processing of waivers pursuant to Section 2697, to facilitate the implementation of this chapter.

(b) In developing the policies and criteria pursuant to subdivision (a), the board shall consult with and consider the recommendations of an advisory committee, appointed by the board in consultation with the commission, composed of the following members:

(1) An engineering geologist registered in the state.

(2) A seismologist.

(3) A civil engineer registered in the state.

(4) A structural engineer registered in the state.

(5) A representative of city government, selected from a list submitted by the League of California Cities.

(6) A representative of county government, selected from a list submitted by the County Supervisors Association of California.

(7) A representative of regional government, selected from a list submitted by the Council of Governments.

(8) A representative of the insurance industry.

(9) The Insurance Commissioner.

All of the members of the advisory committee shall have expertise in the field of seismic hazards or seismic safety.

(c) At least 90 days prior to adopting measures pursuant to this section, the board shall transmit or cause to be transmitted a draft of those measures to affected cities, counties, and state agencies for review and comment.

2696. (a) The State Geologist shall compile maps identifying seismic hazard zones, consistent with the requirements of Section 2695. The maps shall be compiled in accordance with a time schedule developed by the director and based upon the provisions of Section 2695 and the level of funding available to implement this chapter.

(b) The State Geologist shall, upon completion, submit seismic hazard maps compiled pursuant to subdivision (a) to the board and all affected cities, counties, and state agencies for review and comment. Concerned jurisdictions and agencies shall submit all comments to the board for review and consideration within 90 days. Within 90 days of board review, the State Geologist shall revise the maps, as appropriate, and shall provide copies of the official maps to each state agency, city, or county, including the county recorder, having jurisdiction over lands containing an area of seismic hazard. The county recorder shall record all information transmitted as part of the public record.

(c) In order to ensure that sellers of real property and their agents are adequately informed, any county that receives an official map pursuant to this section shall post a notice within five days of receipt of the map at the office of the county recorder, county assessor, and county planning commission, identifying the location of the map and the effective date of the notice.

2697. (a) Cities and counties shall require, prior to the approval of a project located in a seismic hazard zone, a geotechnical report defining and delineating any seismic hazard. If the city or county finds that no undue hazard of this kind exists, based on information resulting from studies conducted on sites in the immediate vicinity of the project and of similar soil composition to the project site, the geotechnical report may be waived. After a report has been approved or a waiver granted, subsequent geotechnical reports shall not be required, provided that new geologic datum, or data, warranting further investigation is not recorded. Each city and county shall submit one copy of each approved geotechnical report, including the mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of its approval of the report.

(b) In meeting the requirements of this section, cities and counties shall consider the policies and criteria established pursuant to this chapter. If a project's approval is not in accordance with the policies and criteria, the city or county shall explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval.

2698. Nothing in this chapter is intended to prevent cities and counties from establishing policies and criteria which are more strict than those established by the board.

2699. Each city and county, in preparing the safety element to its general plan pursuant to subdivision (g) of Section 65302 of the Government Code, and in adopting or revising land use planning and permitting ordinances, shall take into account the information provided in available seismic hazard maps.

2699.5. (a) There is hereby created the Seismic Hazards Identification Fund, as a special fund in the State Treasury. Notwithstanding Section 13340 of the Government Code, the moneys in the fund are continuously appropriated to the division for the purposes of this chapter.

(b) Notwithstanding Section 5001 of the Insurance Code, one-half of 1 percent of the earthquake surcharge moneys received by the California Residential Earthquake Recovery Fund in any calendar year shall be transferred to the Seismic Hazards Identification Fund for the purposes of carrying out this chapter. This subdivision shall become operative only if Assembly Bill 3913 or Senate Bill 2902 of the 1989-90 Regular Session of the Legislature is enacted and takes effect.

2699.6. This chapter shall become operative on April 1, 1991.

APPENDIX B

SEISMIC HAZARDS MAPPING REGULATIONS

CALIFORNIA CODE OF REGULATIONS

Title 14. Natural Resources

Division 2. Department of Conservation

Chapter 8. Mining and Geology

ARTICLE 10. SEISMIC HAZARDS MAPPING

3270. Purpose

These regulations shall govern the exercise of city, county and state agency responsibilities to identify and map seismic hazard zones and to mitigate seismic hazards to protect public health and safety in accordance with the provisions of Public Resources Code, Section 2690 et seq. (Seismic Hazards Mapping Act).

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(1)and (3)-(5)

3271. Definitions

(a) "Acceptable Level" means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project.

(b) "Lead Agency" means the city, county or state agency with the authority to approve projects.

(c) "Registered civil engineer" or "certified engineering geologist" means a civil engineer or engineering geologist who is registered or certified in the State of California.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Sections 2690-2696.6

3722. Requirements for Mapping Seismic Hazard Zones

(a) The Department of Conservation, Division of Mines and Geology, shall prepare one or more State-wide probabilistic ground shaking maps for a suitably defined reference soil column. One of the maps shall show ground shaking levels which have a 10% probability of being exceeded in 50 years. These maps shall be used with the following criteria to define seismic hazard zones:

(1) Amplified shaking hazard zones shall be delineated as areas where historic occurrence of amplified ground shaking, or local geological and geotechnical conditions indicate a potential for ground shaking to be amplified to a level such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

(2) Liquefaction hazard zones shall be delineated as areas where historic occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

(3) Earthquake-induced landslide hazard zones shall be delineated as areas where Holocene occurrence of landslide movement, or local slope of terrain, and geological, geotechnical and ground moisture conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

(b) Highest priority for mapping seismic hazard zones shall be given to areas facing urbanization or redevelopment in conjunction with the factors listed in Section 2695(a)(2)(A), (B), (C) and (D) of the Public Resources Code.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(1)

3723. Review of Preliminary Seismic Hazard Zones Maps

(a) The Mining and Geology Board shall provide an opportunity for receipt of public comments and recommendations during the 90-day period for review of preliminary seismic hazard zone maps provided by the Public Resources Code Section 2696. At least one public hearing shall be scheduled for that purpose.

(b) Following the end of the review period, the Board shall forward its comments and recommendations, with supporting data received, to the State Geologist for consideration prior to revision and official issuance of the maps.

Authority cited: Public Resources Code Section 2696
Reference: Public Resources Code Section 2696

3724. Specific Criteria for Project Approval

The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:

(a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.

(b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:

(1) Project description.

(2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.

(3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.

(4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.

(5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

(c) Prior to approving the project, the lead agency shall independently review the geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2695(a)(3)(A), (B), and (C)

3725. Waivers of Geotechnical Report Requirements

For a specific project, the lead agency may determine that the geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required. This finding shall be based on a report presenting evaluations of sites in the immediate vicinity having similar geologic and geotechnical characteristics. The report shall be prepared by a certified engineering geologist or register civil engineer, having competence in the field of seismic hazard evaluation and mitigation. The lead agency shall review submitted reports in the same manner as in Section 3724(c) of this article. The lead agency shall also provide a written commentary that addresses the report conclusions and the justification for applying the conclusions contains in the report to the project site. When the lead agency makes such a finding, it may waive the requirement of a geotechnical report for the project. All such waivers shall be recorded with the county recorded and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver.

Authority cited: Public Resources Code Section 2695
Reference: Public Resources Code Section 2697(a)(5)

APPENDIX C

TECHNICAL TERMS AND DEFINITIONS

ASTM	American Society for Testing and Materials
CPT	Cone Penetration Test (ASTM D3441-94).
CSR	Cyclic stress ratio— a normalized measure of cyclic load severity, expressed as equivalent uniform cyclic deviatoric load divided by some measure of initial effective overburden or confining stress.
CSR_{eq}	The equivalent uniform cyclic stress ratio representative of the dynamic loading imposed by an earthquake.
CSR_{liq}	The equivalent uniform cyclic stress ratio required to induce liquefaction within a given number of loading cycles [that number of cycles considered representative of the earthquake under consideration].
DSHA	Deterministic seismic hazard analysis
FS	Factor of safety— the ratio of the forces available to resist failure divided by the driving forces.
Ground Loss	Localized ground subsidence.
k	Seismic coefficient used in a pseudo-static slope stability analysis
Liquefaction	Significant loss of soil strength due to pore pressure increase.
N	Penetration resistance measured in SPT tests (blows/ft).
N_1	Normalized SPT N-value (blows/ft); corrected for overburden stress effects to the N-value which would occur if the effective overburden stress was 1.0 tons/ft^2.
$(N_1)_{60}$	Standardized, normalized SPT-value; corrected for both overburden stress effects and equipment and procedural effects (blows/ft).

PI	Plasticity Index; the difference between the Atterberg Liquid Limit (LL) and the Atterberg Plastic Limit (PL) for a cohesive soil. [PI(%) = LL(%) - PL(%)].
PSHA	Probabilistic seismic hazard analysis
$\mathbf{q_c}$	Tip resistance measured by CPT probe (force/length^2).
$\mathbf{q_{c,1}}$	Normalized CPT tip resistance (force/length^2); corrected for overburden stress effects to the q_c value which would occur if the effective overburden stress was 1.0 tons/ft^2.
Sand Boiling	Localized ejection of soil and water to relieve excess pore pressure.
SPPV	Simple prescribed parameter values
SPT	Standard Penetration Test (ASTM D1586-92).
UBC	The Uniform Building Code, published by the International Conference of Building Officials (ICBO, 1997), periodically updated.

APPENDIX D

SOURCES OF STRONG-MOTION RECORDS

California Department of Conservation
Division of Mines and Geology
Strong Motion Instrumentation Program
801 K Street, MS 13-35
Sacramento, CA 95814-3531
    Telephone: (916) 322-3105
    Fax: (916) 323-7778
    http://www.consrv.ca.gov/dmg/

U.S. Geological Survey
National Strong-Motion Program
345 Middlefield Road, MS-977
Menlo Park, CA 94025-3591
    Telephone: (415) 329-5623
    Fax: (415) 329-5143
    E-mail: celebi@usgs.gov
    URL: http://agram.wr.usgs.gov/

Lamont-Doherty Earth Observatory of Columbia University
Route 9 West
Palisades, NY 10964
    Telephone: (914) 365-8477
    Fax: (914) 365-8150
    E-mail: barstow@ldeo.columbia.edu
    URL: http://www.ldeo.columbia.edu/

National Geophysical Data Center
NOAA Code E/GC1
    Attention: Paula Dunbar
325 Broadway
Boulder, Colorado 80303
    Telephone: (303) 497-6084
    Fax: (303) 497-6513
    E-mail: info@mail.ngdc.noaa.gov
    URL: http://www.ngdc.noaa.gov/seg/hazard/strong.html

APPENDIX E

GEOLOGIC ENVIRONMENTS LIKELY TO PRODUCE EARTHQUAKE-INDUCED LANDSLIDES

Landslide Type	Type of Material	Minimum Slope	Remarks
Rock falls	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	40° 1.7:1	Particularly common near ridge crests and on spurs, ledges, artificially cut slopes, and slopes undercut by active erosion.
Rock slides	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	35° 1.4:1	Particularly common in hillside flutes and channels, on artificially cut slopes, and on slopes undercut by active erosion. Occasionally reactivate preexisting rock slide deposits.
Rock avalanches	Rocks intensely fractured and exhibiting one of the following properties: significant weathering, planes of weakness dipping out of slope, weak cementation, or evidence of previous landsliding.	25° 2.1:1	Usually restricted to slopes of greater than 500 feet (150 m) relief that have been undercut by erosion. May be accompanied by a blast of air that can knock down trees and structures beyond the limits of the deposited debris.
Rock slumps	Intensely fractured rocks, preexisting rock slump deposits, shale, and other rocks containing layers of weakly cemented or intensely weathered material.	15° 3.7:1	
Rock block slides	Rocks having conspicuous bedding planes or similar planes of weakness dipping out of slopes.	15° 3.7:1	

Landslide Type	Type of Material	Minimum Slope	Remarks
Soil falls	Granular soils that are slightly cemented or contain clay binder.	40° 1.7:1	Particularly common on stream-banks, terrace faces, coastal bluffs, and artificially cut slopes.
Disrupted soil slides	Loose, unsaturated sands.	15° 3.7:1	
Soil avalanches	Loose, unsaturated sands.	25° 2.1:1	Occasionally reactivate preexisting soil avalanche deposits.
Soil slumps	Loose, partly to completely saturated sand or silt; uncompacted or poorly compacted manmade fill composed of sand, silt, or clay, preexisting soil slump deposits.	10° 5.7:1	Particularly common on embankments built on soft, saturated foundation materials, in hillside cut-and-fill areas, and on river and coastal flood plains.
Soil block slides	Loose, partly or completely saturated sand or silt; uncompacted or slightly compacted manmade fill composed of sand or silt, bluffs containing horizontal or subhorizontal layers of loose, saturated sand or silt.	5° 11:1	Particularly common in areas of preexisting landslides along river and coastal flood plains, and on embankments built of soft, saturated foundation materials.
Slow earth flows	Stiff, partly to completely saturated clay and preexisting earth-flow deposits.	10° 5.7:1	
Soil lateral spreads	Loose, partly or completely saturated silt or sand, uncompacted or slightly compacted manmade fill composed of sand.	0.3° 190:1	Particularly common on river and coastal flood plains, embankments built on soft, saturated foundation materials, delta margins, sand dunes, sand spits, alluvial fans, lake shores and beaches.
Rapid soil flow	Saturated, uncompacted or slightly compacted manmade fill composed of sand or sandy silt (including hydraulic fill earth dams and tailings dams); loose, saturated granular soils.	2.3° 25:1	Includes debris flows that typically originate in hollows at heads of streams and adjacent hillsides; typically travel at tens of miles per hour or more and may cause damage miles from the source area.
Subaqueous landslides	Loose, saturated granular soils.	0.5° 110:1	Particularly common on delta margins.

Modified from Keefer (1984).